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ORDINARY MEETING.

10 January, 1939.

WILLIAM JAMES EAMES BINNIE, M.A., President, in the Chair.

The Scrutineers reported that the following had been duly elected as

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William Chuter Young.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5203.

"The Conditions of Engineering Contracts." †

By Edward Johnson Rimmer, M.Eng., B.Sc., Assoc. M. Inst. C.E., Barrister-at-Law.

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Introduction.

The Council of The Institution gave the Author a very wide discretion in the choice of the title and scope of this Paper. His choice was a difficult one. He had either to take some very limited subject and to deal with it in detail, or to attempt a more comprehensive survey of the whole subject of Engineering Contracts. After much thought he selected the latter alternative, for the reason that exhaustive consideration of any individual subject appeared to him to be more appropriately dealt with in a text-book than by way of a Paper and Discussion. No consideration of the many cases dealing with the subject can be of any material value unless they are treated in detail in the manner adopted in the late Mr. A. A. Hudson's famous book.*

The subject-matter of an engineering contract is generally such as necessitates that the documents of which the contract is composed must make provision for contingencies and events of a special nature, and it is chiefly in this respect that it has peculiarities not to be found in other forms of contract, and is often inevitably of considerable length. Thus the facts that contract works are to be constructed in or erected and fixed on to land, and cannot be rejected and sent back to the Contractor if they prove to be unsatisfactory; that the works are to be carried out in open air under unstable conditions with material and labour of varying quality;

† Correspondence on this Paper can be accepted until the 15th May, 1939.—Sec. Inst. C.E.

* "The Law of Building and Engineering Contracts and of the Duties and Liabilities of Architects, Engineers, and Surveyors." Sixth edition, by A. Inman and L. Mead. London, 1933.

that the conditions of excavation and foundation cannot be entirely foreseen until the ground is opened up ; that execution of the works may result in damage to property belonging to other persons ; that works of specialists may have to be carried out concurrently with work done by the general contractor ; that the period of the contract may extend over several years and the Employer may desire the use of completed parts of the work before final completion of the whole ; and that the amount of money involved is often such as to imperil the financial resources of a contractor who has made an unwise tender, necessitate that terms should be inserted in engineering contracts which would be superfluous to ordinary commercial contracts of purchase and sale.

It may, therefore, be said that, while every contract depends upon its terms (whether express or implied), the engineering contract usually depends in a far greater degree than any other upon the terms expressed in the documents which constitute it, and there remains comparatively little room for legal implication.

For this reason the great volume of case law which has been so carefully analyzed in Mr. Hudson's book must be applied to any particular contract with the very greatest caution, and consideration and discussion by The Institution of case law, or even of the detailed terms of particular clauses, would be almost fruitless.

The law relating to an engineering contract is the general law of contract applied to the particular express terms of that contract. Almost invariably the contract is formal in its nature and consists of several documents, and it is to the terms of those documents, read as a whole, that the Courts will look to define and determine the obligations of both parties ; case law decided on different conditions will not as a rule assist the Judge or an arbitrator in coming to his decision. It is therefore the general nature of the documents which constitute an engineering contract, and their purpose and type rather than their precise terms, that the Author has chosen as his subject for this Paper, under the comprehensive title " The Conditions of Engineering Contracts."

Engineering Contracts.

It is common knowledge that the documents which usually constitute an engineering contract are the following :—

(A) the Contract Drawings prepared by the Engineer ;

(B) the Specification prepared by the Engineer ;

(C) a Bill of Quantities or a Schedule of Prices, or both, prepared by the Engineer or by a Quantity Surveyor, and priced by the Contractor ;

(D) the General Conditions of Contract. The preparation of these is usually the joint work of the Engineer and the legal adviser of the Employer ;

(E) a formal Deed prepared by the legal adviser of the Employer for the purpose of incorporating in one formal document the foregoing documents (A), (B), (C), and (D).

It is now becoming usual to combine documents (D) and (E), as it is quite unnecessary that they should be separate.

In passing it is to be noted (and the circumstance is not unimportant) that these documents may not always be prepared by the same person or at the same time.

(A) *The Contract Drawings.*

These are the Engineer's diagrammatic or pictorial description of the works which, at the time of going to tender, the Engineer anticipates will be carried out under the contract. Although these drawings represent, in the pre-contract relationship of the Engineer and his client, the documents which must be regarded as the most important, they sometimes take a very subsidiary place to the other documents which constitute the engineering contract entered into between the Employer and the Contractor.

The reason for this is that the contract drawings are based upon the knowledge of the Engineer at the time it is determined to carry out the works, before the ground is opened and before all the conditions affecting the carrying out of the work have been completely explored, or indeed have became capable of ascertainment. For this reason it should be provided by the General Conditions that the contract drawings may be amplified, varied, or modified as the Engineer thinks necessary to meet the precise nature of soil, of situation, and of other circumstances—a matter of importance to be referred to in detail later.

It is only necessary to illustrate this by reference to the normal contract drawings issued in connexion with a pipe-laying contract, where the line of pipes is shown perhaps in plan on an Ordnance map (together with longtitudinal sections showing depths and gradients) along public roads and through residential areas. The lines and depths of pipes may, in consequence of circumstances discovered when the work is set out for execution, be varied in regard to position, depth, and gradient, nature of surface, the proximity of buildings (possibly erected between the preparation of the drawings and the execution of the works), etc., and, as a result, drawings of the works as executed might be quite different from the contract drawings.

These same considerations do not apply in the same degree where the work to be executed is solely on the land of the Employer, and certainly in that case there is not the same reason for wholesale variation of the contract drawings. In every case, however, some variation of the work may be, and often is, necessitated, either by the nature of the sub-soil or the sub-soil conditions proving to be different from those which the

Engineer anticipated when he prepared his drawings, or by the Engineer or the Employer desiring modifications to be made in the design, either for the purpose of improving the form of construction, or for economy or for some other cause.

It is for these reasons that the contract drawings may become subordinate to other documents in the contract. It must, however, be pointed out and emphasized that no Court interpreting the contract will readily disregard the fact that the contract was entered into on the basis that the works as shown on the contract drawings were substantially those which the parties expected to be carried out and (unless expressly stated to the contrary) were the works for which the Contractor made his tender, calculated his on-costs and generally arrived at his contract prices.

Accordingly it must be emphasized that the nearer the contract drawings can be prepared to represent the work as eventually carried out, the less likelihood there is of disputes arising under the contract.

(B) *The Specification.*

Subsequent to or concurrently with the preparation of the drawings, the Engineer prepares a specification of the works shown upon the contract drawings. This document should not only particularize the materials to be used and the manner in which the work is to be carried out, but should also set forth clearly the obligations of the Contractor to do or to refrain from doing that which, in the opinion of the Engineer, is respectively desirable or undesirable for the successful carrying out of the work. The fact that the specification does not prescribe the precise methods to be adopted in the carrying out of the works does not, of course, free the Contractor from his obligation to use effective and proper methods which will ensure the completion of the works as they are described, but one of the most serious considerations which an Engineer must give to the preparation of his specification is the extent to which it is necessary or desirable to prescribe methods of carrying out the works.

Generally speaking, it is not considered desirable to specify any description of the temporary means to be adopted by the Contractor in the execution of the works, or the nature of the plant which he is to use. These are matters of which—again generally speaking—the Contractor is best qualified to judge, and his experience and ingenuity in this respect may result in very substantial saving to the Employer in cost. If the use of any particular means or any special plant are not specified the Contractor would, *prima facie*, be entitled to use whatsoever plant and method he might think fit in order to attain the desired result in the most economical manner.

If, however, such means are not specified it may be desirable that the Engineer by his specification should take some overriding control of methods, or at least a right to veto methods which in his opinion are either prejudicial to the ultimate results or may involve danger to adjoining

property or to the works themselves. In specific cases it may be of importance that temporary means of construction should be definitely particularized as being the only method which the Engineer thinks will bring about a successful result. Exceptions to the rule that methods are not usually specified are to be found in relation to processes which research or practice has demonstrated to be the only ones by which the best results are obtainable. It may also be that the design of the work is such that, unless a particular method of construction is followed, parts of the work would be insufficiently strong. It should, however, be realized that the prescription of methods may not only affect the Contractor's tender, but, quite properly, may give grounds for a claim for increased payment if the methods on which the Contractor has been asked to tender are subsequently varied in the light of after-experience.

Upon the clarity with which Engineers are able to express their requirements may depend much of the successful outcome of an engineering contract, and time spent on making a specification as precise and unambiguous as possible is time very well spent in the interests of the client.

(C) *The Bill of Quantities and Schedule of Prices.*

In the Report of the Committee on Engineering Quantities to The Institution [1] (to which special attention is directed under this heading) it is said that the expressions "Bill of Quantities" and "Schedule of Prices" are now used indiscriminately to mean the same thing, and the Committee then proceed to express preference for the expression "Bill of Quantities." In many contracts to-day there is advantage in prescribing a schedule of prices which has no direct concern with the bill of quantities. The bill of quantities is a list or analysis of the items of work necessary to achieve the execution of the works which the Engineer anticipates will be carried out under the contract, and, as the name denotes, the bill purports to show the quantity of units comprised under each item, so that the bill as a whole represents the quantities of detailed work necessary to produce the desired result of the complete contract works. It is true that most bills of quantities comprise items which are provisional, and as such they may represent work in excess of the work shown upon the contract drawings, but they are none the less the quantities of work which the Engineer anticipates generally will or may be carried out. At the same time there may be schedules of items of alternative works, of plant charges and of labour which cannot or ought not to be regarded as part of the bills of quantities, in that they are merely prepared to obtain a unit price from the Contractor in the event of variation, and are not brought into the summary of the bill or into the total tender price at all. For this reason they are not really any true part of a bill of quantities, but when priced would be more properly included in independent schedules of

[1] The Inst. C.E., 1933.

prices. The Author ventures to think that if Engineers would universally adopt the phrases " bill of quantities " and " schedule of prices " as having different meanings instead (as the Committee suggested) being labelled with the same meaning, some of the difficulties which arise in interpreting contracts would be overcome.

What is the purpose of a bill of quantities ? When priced and totalled it should give the total price (which may be called the " contract price ") at which the Contractor has made an offer and at which the offer has been accepted by the Employer. If the bill of quantities has been prepared in accordance with the Committee's Report on Engineering Quantities it will presumably have served the important purpose of telling the Tenderers the number of units of the various items of work which a bill of quantities should comprise that he must allow for in making his tender, and it will thus have saved each of the Tenderers from the unnecessary expense of getting out quantities for himself. So far so good ; but suppose that the quantities are either too full or too lean, or suppose that they omit items of work which it is necessary to execute in order to produce the works shown upon the contract drawings and described in the specification. What then are to be the liabilities or rights of the Contractor and Employer in regard to the adjustment of the contract price ?

The answer to this question must depend upon whether by the General Conditions of Contract the contract proves to be :—

(1) A lump-sum contract whereby the Contractor takes the advantage or disadvantage of the quantities proving too full or too lean.
(2) A lump-sum contract which makes provision for the rectifying of errors in the bill of quantities and for adjustment of the contract price accordingly.
(3) A measurement contract under which all work executed by the Contractor is to be measured on completion and valued at the rates or prices inserted in the bills of quantities.

In the first case the only adjustment of the contract price which either the Contractor or Employer can demand are

(i) the deduction of the " contingency " item ;
(ii) adjustments of provisional items ;
(iii) adjustments due to variations of the works from those shown on the contract drawings.

In the second case either Contractor or Employer can demand, in addition to the aforesaid adjustments, an adjustment resulting from the quantities being too lean or too full, and under some contracts, a varied price for work incorrectly described in the bill. In both the first and second cases the contract price is the sum from which all adjustments are made.

In the third case the contract price is of no account in calculating the payment to be made to the Contractor. He is entitled to be paid for the

work as executed at the prices or rates in the bill of quantities. It matters not in this case if the quantities are lean or full.

A very important feature of all bills of quantities must, however, be pointed out. The preliminary items which are set forth in them may, if the Contractor so desires, be priced as lump sums rather than spread over his prices generally. These items are not quantified, and therefore cannot be a matter for measurement even in a measurement contract. A question often arises whether these preliminary items should be adjusted in proportion as the work done is in excess of or less then the work set forth in the bill. This matter should be made clear in the conditions.

It is also to be noted that within the bill of quantities there may be individual items which the Engineer prefers to be dealt with by way of a lump-sum price rather than by itemized and quantified items. Such lump sum can only be adjusted if the work for which it is quoted is varied. Otherwise, even in a measurement contract, this lump sum is payable and is the total that the Contractor can claim, however excessive or deficient it should prove to be. For these reasons it is of the highest importance that lump-sum prices within the tender total price should be compared by Engineers before they recommend an acceptance of a tender.

In either a lump-sum contract or a measurement contract the prices inserted by the Contractor in the bill of quantities serve as a basis upon which additional and modified work is to be valued and the prices at which omissions are to be deducted ; further, as has been pointed out, additional schedules of work to be priced by the Contractor may be added to the bill of quantities to serve as schedules of prices for varied work not similar to that contained in the bill of quantities. One form of schedule of prices which had its origin in pipe-laying contracts cannot, however, be regarded as at all satisfactory ; that is, the schedule under which a Contractor is required to fill in prices per linear yard for pipes at different depths for any kind of surface or condition which is encountered, and to carry those prices into the bills of quantities where the location of each particular length is described. Nothing could be less scientific. Such prices can only be prices for the average conditions met with, and as every length (whether varied or not) must have some conditions peculiar to it, a valuation based on average conditions of the original scheme may prove unreliable and even ridiculous, and may lead to dispute in the event of an extension of the pipe-line in a location where the conditions of work are worse than the average.

If the real intention of this schedule is to provide for pricing work which proves to be of different depth from that described in the bill, then the Author ventures to suggest that a schedule of percentages representing the increase or decrease required for deeper or shallower sewers than those at anticipated depths is a more suitable provision. Such schedule of percentages could then be used to increase or decrease any price in the bill should the depth of the work for which the price was inserted be varied.

(D) *General Conditions of Contract.*

This subject is a very wide one. It is first desirable to consider the General Conditions in relation to the other contract documents. Even if the specification and the bills of quantities fully define the works to be executed for the contract price, there remains a great number of matters which must be dealt with by the General Conditions. The more these conditions can be standardized the better for the profession and the industry, because a standard form would have the enormous advantage of enabling Contractors and Resident Engineers alike really to understand before work is commenced what are the relative rights of the Contractor, the Employer, and the Engineer. To-day work is carried out under such a variety of general conditions that it is impossible for the parties and the administrators to know precisely where they stand if any dispute arises or appears likely. It is common knowledge in the profession and industry that it is only when a dispute does appear that the conditions are closely considered, and it says much for those concerned that they can proceed so successfully, yet so unscientifically, to execute works for huge sums without either party fully appreciating the rights and obligations which are theirs. There is a strong body of opinion that a standardized form of contract may not prove satisfactory in every case, and, if the specification is to be regarded as subordinate to the General Conditions on all matters where there is divergence, the Author respectfully agrees. A standardized form of General Conditions of Contract would, however, presumably make the specification supreme in regard to all matters respecting obligations of the Contractor to do or to refrain from doing work, and if this were done the Engineer, by his specification, would be able to make provision for all obligations or duties relative to any particular work.

The subject of the General Conditions can, for the purpose of this Paper, best be divided under the broad sub-headings which might well appear in any well-drafted conditions (with certain small modifications these sub-headings were, it is believed, first adopted by the late Mr. A. A. Hudson, K.C.): interpretation and definitions; restrictions against assignments and sub-letting; extent and scope of contract; drawings; general obligations of contractor; labour; work, materials, and plant; commencement, time, and delays; maintenance and defects; alterations, additions, and omissions; property in materials and plant; claims-measurement and valuation of works; prime cost and provisional sums; certificates and payment; remedies and powers; determination of disputes.

Each of these main divisions may consist of several clauses, and it may be necessary to have a main sub-division "miscellaneous." It is impossible in this Paper to deal fully even with the main divisions, but their titles serve as a synopsis of the subject, and by taking them seriatim the Author proposes to draw attention to such matters as appear to him to be of especial importance.

Interpretation and Definition.—Under this heading there is always to be found a definition of "the Engineer," which should be in such a form as will enable the Employer to appoint someone else in the place of the Engineer who is named. This is essential, as in the course of a long contract events may prevent the named Engineer from acting, and as many of the powers of control can only be exercised by "the Engineer" it is necessary that he should survive the period of the contract. Similarly, under this main heading it is usual to define the duties of the Resident Engineer and to state what several documents are to constitute the contract. The enlarging of the definition clause so as to define words and phrases used in the contract should be exercised with the greatest caution, particularly when the component documents are prepared by different persons who may use the same words with different meanings.

Restrictions against Assignment and Sub-Letting.—Under this heading it has, until recently, been usual to prohibit assignment and to allow sub-letting only if the previous permission of the Engineer has been obtained. Of late, however, it has become fairly common to provide a form, to be attached to and filled up with the tender, upon which the Contractor declares the names of the firms to which he desires to sub-let portions of the works, and if these firms are approved before the contract is signed the Contractor may sub-let work to these firms without further authority. The sub-letting of work by the Contractor does not establish any contractual relationship between the Sub-Contractor and the Employer and does not relieve the Contractor of any responsibility whatever under his contract. Out of an excess of caution the clauses normally expressly state this, and also often give to the Engineer power to cancel any consent to sub-let given either under this or other clauses. A nice point arises here for the Contractor. He in turn, if he is wise, will only place his sub-contract on the terms, *inter alia*, that the Sub-Contractor shall have no cause of complaint or claim against the Contractor if the sub-contract is cancelled as a result of the Engineer withdrawing permission to sub-let.

Extent and Scope of Contract.—Under this heading it is only necessary to emphasize that it may be that the works constructed will be something quite different from those anticipated by the contract drawings as a result of alterations, additions, and omissions, and that the contract does not only contemplate the construction of the works but also their maintenance for a certain period after completion. Both these matters must be referred to in any clause purporting to define the scope of the contract. These subjects will be considered in this Paper under their appropriate headings.

Drawings.—Under this heading it is usual, *inter alia*, to deal with the supply of drawings to the Contractor, discrepancies between them and the Specification, and (a matter of first-rate importance) the issue of such further drawings and instructions as may appear to the Engineer to be necessary, by which the Contractor must be bound as if they had formed

part of the original contract drawings and in respect of which the Contractor will not be entitled to any extra payment unless the Engineer then so decides. It is important that this last matter should be considered in some detail.

When the Engineer issues a further drawing or an instruction, the Contractor must comply with it and, as has been stated, must be bound by it as firmly as by the original contract drawings; that is to say, he must not only construct the work to the further drawing or instructions, but must also undertake in regard to it exactly the same obligations (safety, damage to adjoining property, maintenance, etc.) as pertain to the works shown in the original drawings.

If the further drawing or instruction is, in the opinion of the Engineer, a variation of the contract drawings or the specification, he will no doubt issue a variation order, and the contract price will be adjusted, as afterwards appears.

If, however, the Engineer does not issue a variation order then the Contractor will not be entitled to any extra payment for the work. The Engineer's ground for refusing to issue a variation order may be (1) that he thinks that the further drawing or instruction is consistent with the original contract drawings and specification, and is merely an explanation of that which is necessary to carry out the work shown on those drawings and in that specification; or (2) that he does not think any extra cost is involved; and if these are the grounds of his refusal, and if he is right in his opinion, then his refusal involves no hardship to the Contractor.

It must, however, be obvious that the question whether or not a further drawing or instruction adds to the obligation or cost of the Contractor is one upon which there can be two quite honest but conflicting opinions, and in framing a clause on this matter the main consideration must be whether or not, in case of a difference of opinion upon this fundamental matter, the Engineer's decision is to prevail without any right of appeal to arbitration—even if at the time the further drawing or instruction is issued the Contractor makes a claim for extra payment.

How far obligations for safety, maintenance, etc., taken by the Contractor on the basis of the contract drawings and specification should without qualification apply where further drawings and instructions have modified the design, is a question which requires consideration, and is referred to hereafter in relation to "Alterations, Additions, and Omissions."

General Obligations of the Contractor.—Under this heading is a group of clauses placing upon the Contractor all the expenses incidental to the carrying out of the work, the risks and liabilities appertaining thereto, the obligation to indemnify the Employer against all damages done as a result of the work and against actions brought in respect of trespass, etc., and the obligation to comply with all Statutes or Regulations relating to the work. (In this group also are usually included provisions in regard to setting out

testing, employment of agent and foreman, lighting and watching, etc., but these latter matters appear to the Author to be more appropriately dealt with in the Specification.)

The general obligations of the Contractor have become fairly common form, and it is considered almost mischievous to suggest that there can be anything unfair or wrong about them. They work very well, it is said, so they must be right!

The only clauses which call for special comment here are those which place upon the Contractor liability for all damage, whether to the works or to adjoining property, done as a result of the carrying out of the work. What does this mean? It is nothing less than an assumption by the Contractor of the risk (among others) that the work shown in the Engineer's original design, or any varied design he likes to make, may prove not to be sufficiently strong, and the further risk that the work so designed may cause damage to property due to the nature of the ground or the position and proximity of property in relation to it. This is such a heavy risk, having regard to the unknown factors of sub-soil, ground-water level, and (particularly in excavation in streets) proximity of pipes, cables, buildings, etc., that it seems surprising that business men have for so long been willing to take it. It appears to the Author that if under the contract the Engineer has full power to order variations of works and extra works at the expense of the Employer, the Contractor might be disposed to take the risk because he anticipates that if danger to the works or to adjoining property is threatened he would receive an extra-works order from the Engineer. The risk is, however, an impossible one for any Contractor to take if the Engineer is not to be free to order alterations of the work and to give extra-works orders if danger does threaten the works or adjoining work, and unless such extra works are (unless necessitated by default of the Contractor) to be paid for by the Employer.

It is therefore in relation to the clauses dealing with alterations that this matter must be further considered. It should, however, be mentioned here that in the form of contract issued by the Association of Consulting Engineers (hereinafter referred to as "the Association contract") a special clause (clause 39) has been inserted which limits the period of time under which the Contractor is liable for damage to works and property, and limits the extent to which the Contractor is liable in regard to trespass and interference with the property of other persons due to the execution of the work. These limitations, so far as they go, will probably meet with general approval.

The Author ventures to suggest, however, that the liability of the Contractor for surface damage (including destruction of trees, crops, etc.) is a subject which should not be left to be determined by the general conditions of the contract, but is one that should be dealt with expressly by the specification applicable to the particular work with which it is concerned, and that some item in the bills of quantities should direct special attention

to this liability, especially if it is known to the Engineer that suc damage is inevitable.

Labour.—Under this main heading are usually included any speci clauses in regard to recruitment, transport and accommodation of labou first-aid stations, latrine accommodation, and provisions relating to fa wages, and to piecework. None of these clauses call for special commen

Work, Materials, and Plant.—Under this heading are to be found th important clauses which enable an Engineer to compel prompt obedienc to his orders in regard to quality of workmanship and materials employe on and in the works, and to efficiency of the plant.

It must be borne in mind that over a long-period contract the financi position of a Contractor might become such as to lead him to mak economies which are inimical to the safety and efficiency of the work and therefore the powers of the Engineer to give orders and to enforc them by employment of other Contractors are natural safeguards for th Employer to require. It is therefore generally recognized that the authorit of the Engineer to require anything to be done or to require the Contracto to refrain from doing anything should be established by the Gener Conditions. The necessity for these clauses is admitted by all concerned They are peculiar to engineering and building contracts. The fact tha the work is being fixed to the soil makes the remedy of rejection of th work alone inadequate. Not only must the Engineer have the righ to reject the work and require it to be taken down and rebuilt, but th Employer must also have the power to step in to enforce his Engineer' rejection and to take down and rebuild the work if the Contractor shoul fail to carry out the orders given to him. Similarly, the Engineer mus have power to order the cessation of use of a particular piece of plant o the suspension of the whole or any particular part of the work if he shoul regard such stoppage as essential to the general requirements of the situa tion as they affect the work or the safety of property.

More controversial is the question whether or not the reasonablenes of the exercise of these powers may be challenged by a claim for extr payment to the Contractor for compliance with those requirements. I the Engineer required nothing to be done which would not be required t satisfy the Specification no claim could, of course, be established, but if th Contractor were to take the view that some rejection of material or work manship or some suspension of the works or plant was unjustified, and i in fact the Engineer's requirement did call for something in excess o those of the specification, then the Contractor should be paid the extr cost involved. Very few contracts, however, enable the action of th Engineer to be challenged, so far as quality of materials and workman ship are concerned, even for the purpose alone—after the work is complete —of a Contractor's claim. This matter will be referred to again unde the subject of arbitration. In regard to suspension of work it is to b noted that some recent contracts do enable the Contractor to recover th

extra cost resulting from a suspension of work which is not due to his default or necessary by reason of the nature of, or for the safety of, the work, but usually the contract provides that extension of time for completion is to be deemed full compensation for any suspension the Engineer may order.

Commencement, Time, and Delays.—Under this heading are to be found those clauses which establish the contract time, and deal with extension of the contract time and the rights of the parties in the event of delay being occasioned in the execution of the work. Broadly speaking, delays may be occasioned by (*a*) causes over which neither party to the contract has any control; (*b*) causes over which the Employer (or his Engineer) has control; and (*c*) causes over which the Contractor (or any Sub-Contractor) has control.

It is generally recognized that delays occasioned by cause (*a*) should give neither of the parties any grounds for complaint or compensation against the other, but that the Engineer should have the power to extend the time so as to keep it alive when these causes operate. It is also customary for the Employer (in settling general conditions of contract) to protect himself from any claim by the Contractor for damage due to delay occasioned by causes (*b*), even though they prove to be damaging to the Contractor. The justification for such protection for the Employer is generally contended to be that to leave open the possibility of a claim for damages against an Employer for delay in failing to give possession of the site, in failing to give (by his Engineer) the necessary instructions to enable the work to proceed, or generally in failing to afford the facilities to which the Contractor should be entitled in order to enable him to perform his contract, is simply "asking for trouble." Hence in many modern contracts is found an exclusion of any rights of claim by a Contractor other than for an extension of time for even the most flagrant delays caused by the Employer.

On the other hand it is the practice to make the Contractor liable for a sum per day or week as liquidated damages for each day or week by which he exceeds the contract time as defined, or as extended by the Engineer to meet circumstances outside his control; that is to say, the Contractor is made liable for delays over which he and his Sub-Contractor have control. Sometimes this liability is offset by a provision for the payment of a bonus if the work is completed before the expiration of the extended contract period. That provision cannot, however, be recommended except in very exceptional circumstances.

A provision which penalizes the Contractor for delays which he could avoid is clearly comprehensible, even though it may be in the Contractor's own interests to get the job done as quickly as possible, but what is less understandable is the exclusion of damages occasioned to the Contractor by the delays of the Employer—except on the somewhat autocratic basis that the Employer is accustomed to dictate his own terms.

On this aspect of the subject recent case law [1] shows the necessity for careful drafting of the extension-of-time clause and for the proper exercise by the Engineer of his powers of extension. The failure by the Engineer to keep the contract time alive will relieve the Contractor of liability for liquidated damages.

It now appears to be recognized under most contracts that an arbitrator should be allowed to revise, in case of dispute, extensions of time given or refused by the Engineer, although in several well-known forms "the Engineer's decision under this clause is final."

Maintenance and Defects.—Under this heading are set forth the obligations of the Contractor after completion of the works for a period known as the maintenance-period, and generally stated as a period of months. Independently of these obligations any failure by the Contractor to carry out the works in accordance with drawings, specifications, and instructions of the Engineer will, generally speaking, amount to a breach of contract, in respect of which the Contractor will be liable in damages. The maintenance-provisions, however, frequently impose an obligation on the Contractor which does not necessarily arise from any failure to carry out his contract obligations and is not the mere expression of a liability for damages, but arises solely from the fact that some defect may occur in, or be occasioned to, the works during the period of maintenance, and is a liability to do the works of making good the defects. Defects in the work can arise from various causes: (*a*) the failure of the Contractor to do the work as he has contracted to do it; (*b*) the failure of the Contractor to do the work properly, although no proof that he has failed in his contractual duties can be shown; (*c*) the failure of the structure notwithstanding that the Contractor has carried out the work to the entire satisfaction of the Engineer and performed all his contractual duties.

It will therefore be seen that the Contractor, for the limited period of the maintenance-clause, may undertake to make good defects in the works not only for matters in respect of which he himself is responsible, but also for matters in respect of which he has neither responsibility nor control, for example design, physical subsidences outside his control, or other similar causes.

The application of the maintenance-clause to defects which arise through a failure of design or specification must necessarily cause Engineers some misgivings; but none the less a clause in respect of maintenance which requires the Contractor to maintain and uphold the works in perfect condition and to make good defects is such that it will not allow the Engineer to certify for payment for the remedial work, even where the design of the Engineer is in fault, without the express authority of the Employer. It is therefore a matter of some importance for Engineers to consider whether or not they should so frame the maintenance-clause as

[1] MILLER *v.* LONDON COUNTY COUNCIL, 151 Law Times, 425.

to enable them to certify for remedial works in cases where they are satisfied that the defect arises because the structure has—although correctly constructed—failed under load. The Association contract (clause 39) attempts some solution of this matter.

Various other important aspects of the maintenance-clause should be mentioned.

The date from which the period of maintenance runs is usually fixed by a certificate of completion. Some clear determination of this date is necessary. If, however, the date is fixed by the completion of the works as a whole, then the maintenance-period for the work which was first commenced may, according to the length of the contract, be very much extended. It is now fairly usual to find provisions for certifying completion of sections of the work, particularly if they are to be occupied and used by the Employer during the time that other sections are in progress, and for applying maintenance-periods for each section separately. Further, an Engineer would sometimes desire to give a certificate which would have the effect of fixing the commencement of the period of maintenance before he feels he can say that every detail of the work has been completed. In several contracts it has been thought desirable to allow the Engineer to give a certificate of completion on " practical completion " instead (as in the majority of contracts) on completion. The former phrase allows a latitude to an Engineer which in special events (for example, the postponement of supply of some unimportant fitting or of performance of some unimportant service) may be of substantial importance in the fair administration of the contract.

Alterations, Additions, and Omissions.—The provisions relating to " Alterations, Additions, and Omissions " should, in the Author's opinion, engage the very special attention of Engineers. These clauses not only deal with the extent to which variations may be ordered, but with the basis upon which the valuation of those variations shall be made. Under the headings " Drawings " and " General Obligations of the Contractor " attention has been drawn to the fact that, while the work which is eventually carried out may be very different from the work shown and described in the contract documents, the obligations of the Contractor for its execution, for its maintenance, and for damage arising out of its erection, apply equally to the varied work as to the work which he contemplated he would be called upon to do. Thus the Contractor can do nothing more than hope that variations will not increase the difficulties and risks of the work. Moreover, from the point of view of the Contractor variation of the works may materially affect the commercial aspect of the matter in several ways. It may delay the work; may render plant brought on to the site for the anticipated works either idle or inappropriate, or both; may change materially the conditions under which the work, as a whole, must be carried out; and may increase or decrease the cost of the temporary works necessary for its execution.

In spite of these considerations it is very rarely that contracts contain a clause which enables the Engineer to take these matters into consideration in determining the addition or deduction from the contract price which shall result from a variation. Many contracts require that the prices inserted by the Contractor in the bills of quantities or schedules of prices (based on the original contract works) are to be applied to the varied work.

Instead of dealing frankly with the position which may arise and allowing the Engineer to revalue on an equitable basis any work which may be affected by changed conditions resulting from variations, some contracts attempt to minimize an increase of the Contractor's risks by providing that variations shall not exceed in value a certain percentage of the contract price. This leads to serious difficulty and is really a most unsatisfactory provision (it is enough to point out that in pipe-laying contracts almost the whole of the work may be varied from that shown upon the contract sections). A more usual provision is that bills of quantities and schedules of prices shall be used for the valuation of variations only when in the opinion of the Engineer they are applicable, and that if none are applicable the Engineer shall fix proportionate rates. This, however, does not really touch the subject of changed conditions of work brought about by variations, although the Engineer sometimes does give effect to the special considerations mentioned in exercising his discretion under a clause of this sort. Tested, however, by the precise terms of the majority of contracts, the Engineer can only do this with the express permission of the Employer, which, if the Engineer is to preserve his independent position, ought not to be necessary. The safeguard to which the Employer is properly entitled is that a Contractor shall not, in the event of variation, obtain for either the varied work or any additional work prices which are in excess of the level of the prices upon which he accepted the tender. This safeguard is one which the conditions should give, and in the Author's opinion it should be possible to give it without the necessity of precluding the Engineer from giving full consideration to the changed conditions brought about by variations, whether as part of the rate to be applied to the varied work or as a revision of any other rate.

Another important aspect of clauses dealing with variations is the question whether or not variation orders may be given, and, in certain cases, should be given, for the purpose of preventing property adjoining the works from being damaged or for the purpose of rendering practicable works which, as specified, have proved impossible or impracticable by reason of sub-soil or other physical conditions proving different from those anticipated. Under many contracts it may be successfully contended by the Employer that, as the Contractor has undertaken to carry out and to maintain the work and has assumed all risks for damages, it is not the function of the Engineer to render the risk less or to cast extra cost upon the Employer by giving a variation order for any such purpose. The

question may even arise where it can be shown that a variation order or the issue of a further drawing has increased the risk of damage to adjoining property (for example, where a pipe-line has been brought nearer to a building or has been deepened). It appears to the Author that the question whether or not variation orders for protective or strengthening works may be given by the Engineer without first seeking the authority of the Employer is one which should be settled affirmatively by the conditions of the contract.

Property in Materials and Plant.—The provisions relating to " Property in Materials and Plant " need no special notice. Their function is to give greater security to the Employer in the event of the Contractor failing to carry out the work or becoming insolvent. Their form is a legal, rather than an engineering, problem. Legal opinion favours the view that the clause which gives the greatest security to the Employer is one which passes the property in the plant and materials to the Employer when brought upon the site, and (in the case of the Contractor not being a Company) which requires the plant to be marked with the Employer's name during the period of construction.

Claims-Measurement and Variation of Works.—Under this heading it is usual to provide for a monthly submission by the Contractor of a statement of his estimate of the value of the work executed to date. The statement will consist of the items in the bills of quantities which have been commenced, with the appropriate number of units executed to date, priced at the rates in the bill, together with a similar statement of units of work and prices under variation orders. Provision will usually be made for the co-operation of Resident Engineer and Contractor's representatives in the measurement of work for interim certificates, and, where the work which is measured is to be put out of sight, for some permanent record of such measurements to be agreed, or, failing agreement, to be determined by the Engineer. In case of lump-sum items in the bill the claim for interim payments can only be made as a proportion of the work executed to the whole.

By the monthly statements for certificate the Contractor is often required to include with his statement every claim which the Contractor considers due from the Employer, whether for work executed, or for damages, or on any other ground whatever, the cause of which is alleged to have arisen during the preceding month, and failure on the part of the Contractor to make any such claim is often made an absolute bar to its being made at any subsequent time.

This is a provision which is very properly intended to safeguard the Employer against unexpected claims being made at a time when the means of checking them have disappeared ; in the form referred to above, however, it goes much further than that, and may, if enforced, deprive the Contractor of a legal right to which, but for this restrictive clause, he would be perfectly entitled. The question whether or not such severe

restriction is necessary will no doubt be considered. It is to be observed that by the Association contract (clause 44) a modification has been made to the effect that the claims which must be put forward each month are only such claims as are concerned with extra or additional work ordered by the Engineer which the Contractor has executed during the preceding month.

The valuation of works for interim certificate does not, as a rule, raise any difficult questions unless the Contractor is pressed for money, but it is to be noted that in the Association contract (in contrast to other forms) the right of a Contractor to ask for arbitration on a question of valuation for interim payment is not precluded.

On the question of valuation of works for final payment the Author suggests that a sufficiently clear statement regarding the basis of the valuation is often omitted. In his opinion a clause inserted under this section of the conditions should state clearly and unambiguously how the final account is to be made out. Should it be by complete remeasurement or should it be by the addition or deduction from the contract price of additions and omissions, as the case may be ? In either case it is important to state precisely what measurements are to be taken and what units are to be used for the purpose of valuing the work. Presumably it is intended that the units of measurement should be those taken for similar work in the bills of quantities, but it is rarely so stated in terms. If the valuation of the works is to be by addition to and deduction from the contract sum, and not by complete remeasurement, it is important that the precise headings under which the additions or deductions should be made should be clearly set forth ; that is to say, variations of quantities or of depths from those set forth in the bills of quantities ; adjustment as a result of authorized variations ; adjustment of provisional sums ; adjustment of prime-cost items ; and deduction of contingency items.

Prime Cost and Provisional Sums.—Under this heading it is usual to emphasize that, while the Engineer may nominate or select the firms for the supply of goods or for the execution of works for which these sums are provided, his doing so shall not establish any contractual relationship between Employer and Sub-Contractor, and that the Contractor shall remain at all times responsible for the payment as well as for the proper discharge of the duties of the Sub-Contractor. Such Sub-Contractor may have tendered to the Engineer on a specification and on conditions which are not entirely consistent with the specification and conditions in the main contract. This may be due to the fact that the Sub-Contractor is not made aware of the precise main contract conditions, or may himself prepare the specification for the work he is to do. If the Contractor is to assume the main contract obligations in respect of the sub-contract works (for example, obligations of time, maintenance, damage to works and property, etc.), it is quite obvious that the Contractor in turn should have the opportunity of taking obligations from the Sub-Contractor which

will ensure indemnity against loss so far as the sub-contract work affects these obligations. Yet it has often been found that Sub-Contractors whose tender to the Engineer has been provisionally accepted either refuse to accept the main contract obligations or will only accept them if their tender price is increased. To avoid this difficulty it is of importance that when the Engineer asks for tenders for work that is the subject of a provisional sum, he should require them to be given on the terms that the Sub-Contractor will enter into a contract with the Contractor upon terms and conditions consistent with the main contract for securing a due performance and maintenance of the work supplied or executed by such Sub-Contractor, and that he shall indemnify the Contractor against claims arising out of the failure by him to fulfil the terms of the main contract so far as they apply to the sub-contract works. In this respect the general conditions of the contract should, in the opinion of the Author, expressly provide that a Contractor shall not be compelled to enter into a sub-contract in respect of which he is required to take the main contract obligations unless the Sub-Contractor is willing to give the aforesaid indemnity.

Certificates and Payments.—Under this heading it is usual to provide for the giving by the Engineer of interim certificates entitling the Contractor to payments on account; a certificate on completion or practical completion (see p. 17) which releases part of the retention-money; a certificate after all maintenance-obligations have been discharged, which releases the balance; and for the payment by the Employer of all such certificates within a specified time. These clauses call for very little comment and are only slightly varied in different forms. Some require or empower the Engineer in giving interim certificates to take into consideration the value of materials brought on to the site and of temporary works. Some exclude either or both. In so far as materials and temporary works become the property of the Employer when brought on the site it may be thought right that they should be paid for, but provisions in regard to these matters are concerned with finance and, whether one form or other is adopted, must be reflected in the contract price. It is desirable that the clauses should expressly entitle the Engineer to correct a mistake in a previous certificate, and it is quite usual to empower the Engineer to refuse to certify if he is dissatisfied with progress or unless a specified value of work has been done since the former certificate.

Under the payment clause it is usual to entitle the Employer to set off any debts due from the Contractor to him either by way of accrued liquidated damages for delay or by any other manner.

Engineers regard themselves in a quasi-judicial position in regard to the giving of certificates for payment. Some doubt was thrown upon the correctness of this view in a recent case*, in so far as it applies to

* Wisbech U.D.C. *v.* Ward, [1927] 2 K. B. 556; [1928] 2 K. B. 1; 138 L. T. 308; 91 J. P. 200; 11 T. L. R. 62.

certifying interim certificates. Whilst it cannot be doubted that th Engineer owes a duty to the Employer not to go outside the provisions o the contract in determining what he should value or the methods o valuation, the Author ventures to express the view that in valuing a provided by the contract an Engineer would be held to act independently whether the valuation was made for purpose of either interim or fina certificate.

Remedies and Powers.—Under this heading are to be found the specia provisions relating to rights of the Employer in certain circumstances t dismiss the Contractor and to complete the works by other means, an emergency powers.

A Common Law justification for determination by an Employer of contract for works would be that the Contractor, by his acts or omission at the time of his discharge, had evinced a clear intention not to b bound by the contract. Such justification would probably arise if withou proper cause a Contractor withdrew his men from the work, or deliber ately did work contrary to the clear and unambiguous directions of th Engineer.

As the Common Law provides a remedy in a case of serious breach c contract by the Contractor, it is to be doubted whether some of the expres provisions usually found in engineering contracts are either necessary c desirable. If it is found by such a clause that the contract has expressl stated the remedies available to the Employer and the method by which contract may be determined even for the most flagrant breaches of cor tract, it may well be held by the Courts that the parties have agreed t substitute the contract remedies for the Common Law remedies, and if th Employer seeks to take his contractual remedies he may well be held t have lost his Common Law ones. For this reason it appears to the Autho to be doubtful if the operation of contractual formalities and remedie should be expressed by the contract to apply to causes (for exampl abandonment of contract) which obviously give the Employer remedies Common Law.

There is now little divergence otherwise than in detail in regard to th procedure laid down by contracts for discharging a Contractor. Generall speaking it is prescribed in the first place that the Engineer should give notice of his opinion that the cause of complaint exists, and that suc notice should specify that if within so many days the Contractor fails t remove the cause of complaint the Employer may, after a further notic exclude the Contractor from the works, may take over such of the pla and materials as he requires, and may complete the work by other mean It is most important that the notice which finally excludes the Co tractor from the work should be that of the Employer and not of th Engineer. It appears to the Author that no Engineer should take th responsibility for this final step.

When the Employer (no doubt after consultation with the Enginee

has removed the Contractor, the Engineer's duties under the clause usually require that the valuation of the works must still be made on the basis of the contract as a means of comparison with the actual cost when completed by other means, the clause providing that the amount due from or to the Contractor on completion shall (subject to other adjustments and claims) be calculated on this footing. In most current forms of contract the basis of final adjustment in case of removal of the Contractor is precisely the same as it would be if the Employer relied upon his Common Law remedy for damages for wrongful repudiation of the contract.

Any money penalty for failure to complete is unenforceable, although if (as in some exceptional contracts) the right of the Contractor to any further payment for work done is expressly negatived by the terms of the clause, the Courts would probably enforce the provision.

In addition to the remedies in connexion with the failure and insolvency of the Contractor, it is desirable that a clause should be inserted by which, in the event of an accident or failure or other event occurring which the Engineer thinks should be urgently dealt with, as a matter of security, the Employer should be entitled to carry out such work or repairs as the Engineer may consider necessary. The Association contract has an excellent example of this type of clause (clause 38).

Determination of Disputes.—Under this heading are to be found a variety of clauses that have given rise to much controversy, not only within but outside the engineering profession.

Under the earlier forms of civil engineering contract, it was generally provided that the Engineer should act as sole arbitrator for the determination of disputes. At some later date a clause was introduced which was described as a " prevention of dispute " clause, the purpose of which was, it appears, to make it impossible for either party to require any formal arbitration proceedings at all by the simple process of imposing the Engineer's arbitrary decision to prevent a dispute arising.

In the course of time the wording of this clause assumed a form which provided that, in order to prevent disputes, whether before commencement, during progress, or after completion of the works by the Contractor, regarding any claims, whether arising under or out of the contract or from breach or alleged breach thereof or in any way incidental thereto or connected therewith, the Engineer's measurement, valuation, decision, or certificate should be final and binding upon the Contractor and the Employer.

It might appear that that was, in fact, merely stating that disputes should be settled by the Engineer, but that is far from being the case. Under such a clause the Engineer was not an arbitrator at all, but was a complete autocrat who was not bound, in forming his decision, to observe any of the statutory provisions dealing with arbitrations or any rules of evidence, or, in fact, to act otherwise than as a dictator.

This clause sometimes even went to the length of stating expressly

that in measuring, valuing, deciding, or certifying as above-mentioned, the Engineer was not intended to act judicially. Even when, as often, this clause was followed by a clause which provided that if there remained any question, dispute, or difference, for a settlement of which provision was not already made, such dispute or difference should be referred to the determination of an independent arbitrator, the full powers of the Engineer were preserved because there could not remain to be referred to the independent arbitrator a dispute on any fundamental matter, so that in effect the result of the prevention-of-dispute clause came to be that no effectual arbitration could take place at all.

The use of this clause is now falling into desuetude, and modern practice is to make provision for all disputes to be referred to the decision of an independent arbitrator, subject to protective provisions to ensure that disputes do not delay the work, and subject also in many cases to the exception from the operation of such a clause of disputes relating to certain matters, in regard to which the decision of the Engineer is expressed to be final and without appeal to arbitration.

In a famous judgement given in 1893 * by Lord Justice Bowen, where the form of contract provided that the Engineer should be sole arbitrator, it was said :—

> "Employers find it necessary in their own interests, it seems, to impose such terms on the contractors whose tenders they accept, and the contractors are willing, in order that their tenders should be accepted, to be bound by such terms. It is no part of our duty to approach such curiously-coloured contracts with a desire to upset them or to emancipate the contractor from the burden of a stipulation which, however onerous, it was worth his while to agree to bear. To do so would be to attempt to dictate to the commercial world the conditions under which it should carry on its business. To an adjudication in such a peculiar reference, the engineer cannot be expected, nor was it intended, that he should come with a mind free from the human weakness of a preconceived opinion. The perfectly open judgment, the absence of all previously formed or pronounced views, which in an ordinary arbitrator are natural and to be looked for, neither party to the contract proposed to exact from the arbitrator of their choice. They knew well that he possibly or probably must be committed to a prior view of his own, and that he might not be impartial in the ordinary sense of the word. What they relied on was his professional honour, his position, his intelligence ; and the contractor certainly had a right to demand that whatever views the engineer might have formed, he would be ready to listen to argument, and, at the last moment, to determine as fairly as he could, after all had been said and heard."

* Jackson *v.* The Barry Railway, [1893], 1 Ch. 238.

Although it is clear, therefore, that the Courts would enforce these "curiously-coloured contracts," a Committee appointed by the then Lord Chancellor in 1926, under the chairmanship of the present Lord Justice Mackinnon, to consider the amendments necessary to the law of arbitration, made a Report which resulted in the passing of the Arbitration Act, 1934. By that Act it is provided that, where an agreement between any parties provides that disputes which may arise in the future between them shall be referred to an arbitrator named or designated in the agreement, and after a dispute has arisen any party applies, on the ground that the arbitrator so named or designated is not or may not be impartial, for leave to revoke the submission or for an injunction to restrain any other party or the arbitrator from proceeding with the arbitration, it shall not be a ground for refusing the application that the said party at the time when he made the agreement knew, or ought to have known, that the arbitrator, by reason of his relation towards any other party to the agreement, or of his connexion with the subject referred, might not be capable of impartiality.

It thus appears that under recent legislation Parliament has gone a long way to counteract the effect of Lord Justice Bowen's decision.

It appears to the Author that under the form of contract used in earlier days it might well have been appropriate to provide that all matters of dispute should be referred to the Engineer, who, untrammelled by any particular loyalty, would do what he thought to be fair in all the circumstances and would certify payment without undue concern for contractual terms, but on the broad basis of fair play all round. In contracts which did not attempt to deal in detail with the various contingencies which might arise in the carrying out of engineering works, such a provision may well have served a useful purpose.

When, however, contracts for such works came to assume a form (no doubt under the hands of legal advisers) which took into express consideration and purported to provide for every contingency, to place restrictions on the revision of prices, to specify the manner in which varied works were to be valued, and to deny expressly to the Engineer the right to certify for losses to the Contractor due to delays outside his control, and similar matters, both the necessity for, and the desirability of, endowing the Engineer with supreme powers ceased to exist. The Engineer in such circumstances was no longer necessarily the best person to exercise judicial functions in connexion with matters of interpretation of or other disputes under such a contract. Moreover, the execution of major engineering works was to a rapidly increasing extent becoming a function of permanent officials of Employing Authorities, officials whose position not only demanded and engendered a special loyalty to their employer, but who might by reason of that position at any time be called upon to account to the Employing Authority for their actions not as consultants, but as employees.

In short, it is not too much to say that the retention in the hands c the Engineer of supreme power as sole arbitrator has, in relation to thl terms and circumstances of present-day contracts, become something c an anachronism. Old forms change slowly, and reform in this respect wa long overdue. Whether the reform, so far as it may be represented by thl Association Contract (clauses 55 and 56), is on entirely right lines is ope to question.

These clauses take the shape of a provision for reference to independen arbitration of all disputed matters with the exception of some reserve t the sole decision of the Engineer. In considering whether this is thl true solution to the problem it is necessary to consider the interests of thl Employer, of the Engineer, and of the Contractor.

The Employer's standpoint is, in effect, that in engineering contract there is so much potential material for disputes, that unless he gives hi Engineer power to make decisions which cannot be appealed against, h may be faced with claims based on a review of such decisions leading t costly litigation or, in order to avoid it, additional payments by hin What he thinks is of paramount importance is that he should have a ba to arbitration or litigation, namely, the final decision of the Engineer.

The Engineer's interest is twofold. He desires that there should b no doubt whatever that the Contractor shall do whatever the Enginee tells him to do in respect of all matters relating to the efficiency an sufficiency of the work. He also wants to be sure that if any claim is t be made as a result of his instructions, he shall receive notice of the natur and grounds of that claim at the time the instruction is given, so that h may, if necessary or desirable, modify it.

The interest of the Contractor is to ensure that if there is a difference o opinion—which is generally a difference between the Engineer and himself— either upon any point of construction of the contract, or as to whethe compliance with an instruction or direction from the Engineer entitles hin to increased payment, such difference is determined by a person uninflu enced by any personal connexion with the contract whatever.

Primarily, therefore, any provision for the determination of dispute should make it clear that during the progress and until completion of th works, the Engineer's decision is paramount and must receive immediat obedience, which satisfies the first of the interests of the Engineer abov referred to.

Secondly, provision must be made which avoids the raising of dispute at the end of a contract of which no previous notice has been given an the evidence to rebut which may have disappeared. The nature of th work comprised in a civil engineering contract is obviously one whic requires special provision in this respect, and the Contractor should there fore be put under some form of obligation to give notice that he will as the arbitrator to review the Engineer's decision at the time the decisio he disputes is given.

A provision which would satisfy both these interests of the Engineer by deferring arbitration to the end of the contract and preventing claims being then made based on the review of an Engineer's decision which the Contractor at the time did not at once state to be unreasonable, and which would satisfy the interest of the Contractor by giving him the right to have his contract remuneration determined by an independent person, might, in the opinion of the Author, be preferable to the exclusion of the right of appeal from the Engineer on specific matters.

Conclusion.

The necessity for keeping this Paper within reasonable bounds does not allow a more detailed consideration of the subject, nor was it thought necessary to refer to the last of the contract documents—the formal Deed—because that, if separate from the General Conditions, is merely a formal legal document. Enough has been said to show that from the general point of view of the engineering profession, of Employers and of Contractors, the time for the adoption of standard Conditions of Contract (subject to qualification by the terms of the specification in individual cases) is overdue. If once the General Conditions were standardized, then, in the opinion of the Author, all special conditions necessary for any particular job could be dealt with by the specification, with references back to it in the bill of quantities.

One important word of warning must, however, be given. The basic principles upon which standard conditions should be framed must first be decided by the parties concerned. They may, amongst persons of different views, be determined on the basis of compromise, but if the conditions are to be clear and unambiguous there is no room for compromise on the manner and phraseology in which these principles are to be stated. This must lead, in the Author's opinion, to the overthrow of some clauses and phraseology now taken to be standard practice, but which, in fact, are confusing and ambiguous.

The Author wishes to acknowledge with thanks the assistance given to him by Mr. Kenneth Thomas, B.A., LL.B., who not only read the draft of this Paper but made valuable suggestions which have been incorporated into it in its final form.

Discussion.

The Author observed that the general conditions of engineering contracts in use in Great Britain at present were the result of the efforts of innumerable draughtsmen, legal and otherwise, of the past, and he thought that it would be clear that the main purpose of the draughtsmen who first settled an engineering contract was to make provision for those contingencies which might arise in the carrying out of such precarious work in such a way that, however far-reaching the modifications of the work and the conditions of the work as a result of those contingencies the basis of the contract should remain; that was to say, that no event however unexpected, should vitiate the contract, and that the contract time should be kept alive by the very simple expedient of an extension of the contract period to meet the contingency and the delays thereby caused.

Gradually, however, the general conditions of contract, being drafted by the legal adviser of the employing authority, in collaboration, it might be, with their consulting engineer, and under the hands of successive advisers, reached a stage when not only had the contract been preserved in any event that might happen in its execution, but the contractor had imposed upon him almost all the risks of loss occasioned by such events. When it was appreciated that the event which caused such loss might be an event for which the Employing Authority themselves were responsible, or an event brought about by a variation in the character of the work ordered by the Engineer, or even by the failure of the structure as designed under load, it followed that the criticisms of general conditions were necessarily almost all in one direction, resulting in a consideration of the question whether or not some amelioration of those risks, and some removal or limitation upon the liabilities to be undertaken by the Contractor, would not be a more satisfactory solution of the problem.

Whilst in the Paper he had generally refrained from expressing any opinion regarding which of two views should prevail on certain aspects of the subject—he had set out those views frankly, so that those who studied the Paper might in turn adopt, or advise the Employing Authority to adopt that course which in their opinion was best suited for the work which it was proposed to carry out—in his reference to some commonly-accepted conditions he had found it very difficult to disguise the fact that in practice he had found them to be one-sided, harsh or even unreasonable; he had shown himself generally in favour of a contract consisting of a standardized General Conditions of Contract together with a Specification which would take precedence of the General Conditions in regard to special matters applicable to the particular contract work.

He ventured to think that a standardized form of General Conditions would have advantages to employing authorities as well as to contractors, and would be of great value to the engineering profession as a whole. Under such standard General Conditions of Contract the respective rights of the parties should be of such a nature that they might be fairly enforced whatever contingencies might arise, and in that event, if they were adopted, it should be understood by all parties that in the event of a dispute arising every clause would be enforced without question. It was common knowledge that employing authorities, and some Engineers, when challenged regarding the application of their General Conditions, had often been heard to say that they imposed certain liabilities or limitations upon the Contractor as a matter of protection only, and not with any intention, except in a bad case, of enforcing them. It had even been suggested that no fair-minded Engineer would ever think of enforcing some of the conditions still to be found in some contracts, but that they were inserted solely for the purpose of dealing with a Contractor who in the opinion of the Engineer acted dishonestly; and Contractors were known to accept conditions which on the face of them would, in the events contemplated, prove very harsh, because they were confident that such conditions would never be enforced by the Engineer concerned.

The Author and his colleagues at the Bar found considerable support for the view that the conditions of engineering contracts were not always enforced. They advised clients that some clause in the General Conditions was a complete bar to a Contractor's claim, but they found that a representation to the Engineer or to the Employing Authority that the clause which stood in the way of the claim was unreasonable often opened the way to the claim being met. In fact, one of his colleagues suggested that he was wasting his time in studying conditions of engineering contracts, because a legal basis was hardly ever necessary upon which to found a Contractor's claim if the Contractor could persuade the Engineer that it would be a hardship to him if the claim were not admitted.

While cases of that kind did great credit to the Engineer's sense of natural justice, the Author would regard the state of things to which he referred as being most unsatisfactory from the point of view of the Engineer. Sometimes the Engineer had to act for a client who expected the contract conditions to be rigidly enforced; if so, what risk might he have run if he had failed to enforce them, or at least what disappointment would he be bound to feel when he had to say to the Contractor that his recommendation to admit a claim had been refused by his client on the ground of a condition of the contract which he (the Engineer) never intended to enforce, and which he had inserted merely as a protection? He could give from his own professional experience many cases which illustrated that state of things, and he therefore suggested that the choice which had to be made was as between a standardized set of fair conditions which would be strictly enforced, and the continuance of the existing practice; as a lawyer, he

would be false to his legal training if he did not favour the first of those alternatives.

If standard conditions were to be used and enforced, then the Paper indicated many matters which would have to have the consideration of those who decided the principles upon which they were to be settled. It was far from exhaustive, as he had been compelled by limitations of time and space to confine it to a few of the many matters which had come to his knowledge during his practice. He trusted also that the Paper would be of value to the younger men of the profession who might, in the light of the observations which it contained, be encouraged to give closer attention to the conditions used in connexion with works upon which they were engaged.

Mr. W. T. Halcrow said that, after showing clearly the essential differences obtaining between a contract for constructional works and one for the supply of goods or machinery, the Author proceeded to a general survey of the conditions governing and affecting a contract of the former kind. Mr. Halcrow found himself so much in agreement with the Paper that he could do little but emphasize some of the points which appeared to him to be of especial importance.

It was his experience that some engineers were apt to regard the General Conditions of Contract as a purely legal document and to think that their preparation was a matter rather for the lawyer than for themselves. His own view was that the Engineer, as the person responsible for the conduct of the work, should decide the basis for the contract between the Employer and the Contractor, and should, therefore, to a large extent prepare the conditions of contract himself. He should, however, obtain legal advice so as to avoid the possibility of imposing any condition which would conflict with the law.

The tendency to regard the conditions of contract as a legal document was illustrated by the fact that he had found resident engineers who had not even troubled to read the conditions of the contract of which they had been in charge. In his view, engineers engaged upon the construction of works should endeavour to understand the importance of the General Conditions.

Matters affecting the specification of temporary works and methods of construction were dealt with on p. 6. If the Engineer specified in detail how the work was to be carried out, the Contractor was left with little scope for the exercise of his skill and experience, and tended to become merely a purveyor of labour and materials. Whilst Mr. Halcrow fully realized that there were works which required special treatment, in his judgement it was better to give the Contractor as much freedom as possible, provided that the stability of the permanent work was not adversely affected. More favourable tenders might result thereby, to the benefit of the Employer.

In measured contracts it sometimes happened that the actual quantities

of work carried out under any item varied considerably from those in the bill, and questions arose at times regarding the fairness of applying the schedule rates to the measured quantities. It seemed to him that the conditions covering that matter should be drawn up to give the Engineer powers which would be as wide as possible, in order to deal with variation in prices in an equitable manner.

Reference was made on p. 13 to the Contractor's liability for damage to adjoining property, resulting from the carrying out of the work. That was often a difficult point, and he believed that the most satisfactory way of dealing with it was to prescribe in the contract a limit to the Contractor's risk in each particular case.

The conditions covering the maintenance of works were also discussed in the Paper. Whilst it would be agreed that the Contractor would have to make good any defects of materials or workmanship, the question of his assuming responsibility for matters over which he had no control was one which required review. Mr. Halcrow did not think that there should be much difficulty in arriving at a clause which would be fair both to the Employer and to the Contractor. The form which was usually found in contracts to-day seemed to leave something to be desired from the point of view of equity. The remarks in the Paper with regard to the date of completion being based upon the substantial completion, instead of on the final conclusion of the work, were also very much to the point. Some engineers had adopted the expression "practical completion" or "substantial completion," which appeared to him to be fair.

On p. 19 the Author dealt with variation orders for protective or strengthening works. In Mr. Halcrow's view, the Engineer should, where the safety of the work was concerned, have the power under the contract to order what was necessary without previous reference to the Employer.

Under the heading "Determination of Disputes," reference was made to the Engineer acting as sole arbitrator as an alternative to the appointment of an independent arbitrator. There had been cases where that had occurred, although he had never been in such a position, nor would he accept it. Whilst the Engineer during the course of the work had necessarily to arbitrate on matters which might from time to time arise, Mr. Halcrow's view was that, the Engineer having been in close contact with the work, it was not easy for him to detach himself from the personal knowledge which he had acquired and to place himself in the position of an independent arbitrator. He was therefore of opinion that provision should always be made for an independent arbitrator.

The arbitration clause, as usually inserted in contracts, provided that the arbitrator's award should be final and binding on all parties, and should not be liable to any exception or objection whatever. When he first interested himself in such matters he thought that the words meant what they read, but apparently they did not; it was possible for certain matters

to go beyond the arbitrator to the courts. He had sometimes thought that, because of the nature of the works being carried out—and that point was very clearly put in the Paper—there should be some attempt to make the decision of the arbitrator final and binding.

The Author's quotation from Lord Justice Bowen's judgement was of interest. Mr. Halcrow had only to turn up old contracts prepared by predecessors in his firm to find cases where the Engineer's powers put him very much in the position of a dictator. The Contractor was apparently not allowed to have any rights, but only liabilities. At the present time, however, the Contractor was, as he ought to be, in a better position, as efforts had been made to limit his liabilities to those matters over which he had personal control. When drawing up conditions of contract it should be the Engineer's duty to ensure as far as possible that both Employer and Contractor would have a square deal.

There was a good deal to be said for the standardization as far as possible of the conditions of contract. The standard conditions drawn up jointly by the Association of Consulting Engineers and the Federation of Civil Engineering Contractors had proved of material value, and there was a steadily increasing demand for copies of them; he favoured any move towards carrying that work a step further.

He was in entire agreement with the Author's remark that the conditions should be drafted so as to be clear and unambiguous. Legal phraseology should be avoided as much as possible, as it was not easily understood by the non-legal mind. The conditions were, or should be, a working tool for those who had to carry them out, and should therefore be readily understood by them. As an engineer, he felt that the simplest language best suited the purpose.

Mr. A. M. McTaggart would like to mention a few points which occurred to him as being important from the point of view of the Contractor. In the majority of cases, for works of a substantial nature, the Contractor had to supply a detailed description with his tender showing how he proposed to carry out the work. That, in Mr. McTaggart's view, was really a contractor's asset, and, by giving such detailed information on a contract in which he might not be successful, he was giving away a great deal of his stock-in-trade. He suggested that a general description showing the intended method of carrying out the works would be sufficient in the first case, a more detailed description, in the event of the tender being considered, being at the disposal of the Engineers at that time.

He would also suggest that more positive information should be given with inquiries, and that details of bore-holes should be more specific so as to eliminate gambling as far as possible. There was sufficient margin in the general layout of a contractor's scheme and in his organization and ingenuity to make tenders competitive without the gambling influence being introduced.

With regard to bills of quantities, it was the habit of some engineers

to over-estimate those quantities so as to leave a margin for contingencies. He suggested that the margin ought to be stated, so that a contractor might measure the fair value of the work; otherwise, if quantities had been over-estimated by, say, 10 per cent., and they came out net, the Contractor was losing on the percentage which he had added to his basic figures to cover plant, stores, contingencies and general overheads, and profit.

Conditions of contract varied to a large extent, depending, apparently, more or less on the clients for whom the work was being carried out. Some clients, through their legal advisers, would appear to draw up what they considered should be the obligations of the Contractor, which in the majority of cases meant that whatever happened the Contractor had to pay. That method of drawing up General Conditions was, he felt sure, not generally acceptable to Engineers, but was rather forced upon them by their clients. He was glad to hear that Mr. Halcrow did not agree with conditions of that type.

Arbitration was a subject which led to all kinds of controversy, especially where the Engineer specified that he was to be the sole arbitrator. As a contractor, he appreciated that the Engineer in some cases felt that he knew the interpretation of what he had written and what he intended better than anyone else, but that point of view was open to criticism, as another person reading the same words and the same clauses might quite well put a different interpretation upon them and give them a different meaning. The Engineer might insist on having the final say on certain subjects in connexion with the work where an arbitrator would find difficulty in getting a true picture, but at the same time, as a general question, independent arbitration was probably preferable to all parties. He was glad to hear that Mr. Halcrow took that view.

There was a time when the Consulting Engineer, if asked why he had put certain things in the specification, would say, "I must protect myself against being landed with the worst type of Contractor and one who may take advantage of any loophole." Times had changed, however, and to-day contracting firms had on their staffs engineers who were as proud of turning out a good job as the Engineer who designed the scheme.

It was not an uncommon practice for Consulting Engineers to insert in the specification a clause making the Contractor responsible for whatever might happen to the works during the construction and maintenance period, including responsibility for failure of design. That, in Mr. McTaggart's opinion, was completely wrong, as surely the Engineers, being the party who had all the data available for the design, should be responsible for it; it appeared to Mr. McTaggart to be very unfair to ask the Contractor to accept responsibility for something which he had had neither the opportunity of studying, nor data on which to base calculations in order to check the Engineers' design. The Contractor's responsibility, should, in Mr. McTaggart's opinion, be confined to workmanship and material.

Mr. Kenneth Thomas observed that the engineering contract was a peculiar document, or compound of documents, inasmuch as there was nothing quite like it in any other sphere of industry or commerce, with the exception of the building industry, and even there an examination would reveal that there was only a very superficial resemblance between the building contract and the engineering contract. The form of the latter, as the Author indicated, was to some degree attributable to the special nature of the work which it covered, but that alone was not enough to explain its apparent one-sidedness and the peculiarly heavy risks and obligations which it placed on one party to the contract, namely the Contractor. He thought that there had for a good many years past been a tendency on the part of certain Employers, and certainly on the part of many Engineers, to recognize that there was a case for the modification of that type of contract in order to adjust the balance a little more evenly between Employer and Contractor; that had resulted in a very slow and gradual improvement in the conditions in individual cases, but taking engineering contracts in the mass throughout the country, and covering works both great and small, they were still subject to the criticism—and that applied particularly to forms of contract in use by local authorities —that they were unduly onerous and harsh to the Contractor, and he ventured to think uneconomically so.

It might be of interest to consider the reasons generally advanced for the continued retention of contracts in such a form, which frequently ran counter even to the natural principles of justice. In his experience, they fell generally into three classes. It was sometimes said, "We must protect ourselves at all costs against the dishonest contractor." On other occasions the plea was, "We must safeguard ourselves against the litigious contractor." Those two reasons, it would be freely acknowledged, were far more frequently advanced by lawyers than by engineers. Thirdly, it was frequently said "We have used these conditions for many years. They have always worked very well; why should we revise or alter them?"

He would like for a few moments to examine those reasons. The first was the danger of the dishonest contractor. There were no doubt black sheep in every walk of life, and in the contracting industry no less than elsewhere, but certainly no more than in any other sphere. There were probably black sheep even in the building industry, but that had not prevented the adoption in that industry of a standard form of contract, the well-known R.I.B.A. form, which did hold an even scale between the Employer and the Contractor, and which was almost universally used throughout the country for works of building construction. The truth was that the employer could be protected against the dishonest contractor just as well by fair and equitable conditions as he could by unfair and harsh conditions, provided that those conditions were properly and wisely drawn; and contractors as little as anybody would deny the advisability of all

reasonable and proper provisions to safeguard the employer against dishonesty.

In regard to the litigious contractor, if ever Mr. Thomas were unfortunate enough to get a contractor into the arbitration-room or into the Court he found that he got a black mark against his name for having failed in what the contractor regarded—and Mr. Thomas agreed—was his duty, which was to keep him out! The truth was that the litigious contractor did not exist, and even if there were a contractor who was litigiously-minded he speedily recognized the damage to his reputation and to his business which would ensue if he indulged himself in so misguided a taste.

The third argument, that the conditions had served very well for many years and that there was no need to change them, did not connote that the clauses in the form in question had been carefully and individually tested in the Courts and found to stand the test; there were very few forms of conditions of contract of which that could be said. It merely meant that the user of such conditions had been fortunate enough to experience little or no trouble in practice, for which there might be many reasons, and he therefore assumed that there was no likelihood of trouble.

The question of standardization had been referred to. His personal belief was that it would confer an immense boon on all those classes of the community who were interested, whether as employers, engineers or contractors, in engineering works. He did not minimize the difficulties attendant on the undertaking of such a task, but he had yet to learn that the difficulties attendant on any problem ever deterred either an engineer or a contractor from tackling it.

There appeared to be four essentials in the devising of a satisfactory standard form. In the first place, it was necessary to segregate those conditions which were really common to all forms of contract, no matter what work they might cover, from those conditions which were bound to vary to some extent according to the nature of the work. The latter had either to be carried to the Specification or else—more helpfully, in his view—incorporated in a supplemental document called Special Conditions; and, since there would still be a good deal of common ground about those last-named conditions, it might be very helpful if model clauses were drafted which would be suitable for use in different circumstances.

Secondly, the General Conditions proper had to be drafted in such a way as to endow the Engineer with the fullest powers to have the work executed as he wanted it. That was of supreme importance, because it was humanly impossible to foresee every contingency in connexion with an engineering contract at the time that the drawings and specifications were prepared. What was of no less importance from another point of view was that the Contractor should be entitled to be paid for his work if the requirements of the Engineer involved him in additional expense. In that connexion, it might be fruitful to examine the question whether or not the Engineer ought not to have power to vary the contract price where the

requirements that he thought it desirable to impose in the interests of the work either greatly increased the Contractor's risks relative to that work, or materially altered the quantities of the work so as to make, in effect, the Contractor's itemized price for that work uneconomical relative to the altered quantity which he had to do.

Thirdly, an effort had to be made to make a fair apportionment of responsibility for risks as between Employer and Contractor. That was a matter on which there was room for wide differences of opinion, and it necessitated a careful and sympathetic examination of the different views. It seemed to him that in that respect the economic aspect would have to carry very great weight, and that the real desideratum was to achieve a basis of tendering which would secure the minimum price being offered to the Employer for the work which was to be carried out.

Finally, there was the all-important matter of draughtsmanship. When the points of principle had been settled as best they might be, the draughtsman had to exercise his skill in the use of language so as to ensure, so far as was humanly possible, that each word and sentence had its proper value and correctly expressed the intention of the authors. The draughtsman's task was, or should be, to eliminate ambiguities, so very frequent a source of dispute, and to achieve a document which was not only standard in form but was as perfect in clarity of language as a standard document ought to be. The authors had, however, necessarily to be at pains to see that the draughtsman, in his enthusiasm for clear and correct expression, had not altered the sense of what the authors of the document meant to say. The task called for exemplary patience and care, for extreme frankness in the expression of intentions, and for the complete abandonment of those methods of subtlety and finesse which were designed to gain for one side or the other some advantage based on someone's failure to see what was being implied. The result would, however, in his opinion be ample reward for all the patience and labour expended upon the preparation.

Mr. Raymond Carpmael remarked that any views which he expressed were not to be taken in any way as committing his company, the Great Western Railway. As he sometimes did when he realized that his personal opinions should have a secure foundation, he had referred to those of his illustrious predecessor in office, Isambard Kingdom Brunel. In the archives of the Great Western Railway he found the following words of Brunel, uttered nearly 90 years ago :—

"You may have seen that a great appeal case before the Lords, affecting claims of some hundreds of thousands, has just been finally decided in favour of the Great Western Railway Company after fourteen years of litigation; and this favourable decision was entirely obtained by carefully prepared specifications, and by my not having departed in any single case, in years of correspondence, from the letter and spirit of the contract, and particularly from the fact—strongly commented upon by Lords Cranworth and Brougham—that I had maintained my position of umpire between the Company and the Contractor. It is, then, as essential to the Company as to the Contractor and to me that I should maintain that position."

The case referred to was that of RANGER *v.* THE GREAT WESTERN RAILWAY COMPANY. Although the Author rightly emphasized the importance of the clarity with which engineers should express their views, Mr. Carpmael ventured to suggest that the spirit was of even greater importance than the letter. It would appear that Brunel himself actually took that view, because it was recorded that in his opinion :—

> " In a railway the only works to be constructed are engineering works, and there can really be only one engineer ; and in your case especially, where, as I apprehend, the contractor is part of the company, and has to be treated with consideration, and perhaps less vigorously, at all events differently from an ordinary contractor, considerable management and discretion will be required of your engineer, and a degree of responsibility which I would only undertake if sole engineer."

The only comment which Mr. Carpmael wished to make on that took the form of a question : Why should ordinary contractors (if there were such people) be treated differently from railway contractors ?

It was a Great Western Railway tradition, which his predecessors and himself had followed, that the Contractor should be treated with consideration, as part of the Company. Harsh and unreasonable treatment of contractors was certain to result sooner or later in an advancement of tender prices as compensation.

On p. 6 the Author emphasized that " the nearer the contract drawings can be prepared to represent the work as eventually carried out, the less likelihood there is of disputes arising under the contract " ; and on p. 7 he said : " Upon the clarity with which Engineers are able to express their requirements may depend much of the successful outcome of an engineering contract, and time spent on making a specification as precise and unambiguous as possible is time very well spent in the interests of the client." Mr. Carpmael would go further and say " in the interests of everybody." He fully agreed with the Author's statement ; his own Company followed the Brunel tradition in taking particular care to make their drawings and specifications as representative and precise as possible, and he had reason to know that that was realized by their contractors.

With reference to the Author's remarks regarding the bill of quantities and schedule of prices, almost all Great Western Railway contracts were " lump sum " and contained a " provisional clause." The amount of the tender therefore equalled the total of the bill of quantities ; the schedule of prices set out the unit prices on which adjustments in the contract quantities were based. With regard to preliminary items, which were referred to on p. 9, Great Western Railway practice provided for separate sums to be stated for " preliminaries," and although no provision was actually made for adjustment thereof, the equity of adjusting some at least of them was apparent in the event of extensive variation of a contract. Such cases were dealt with " in the spirit of the contract." The Author also said : " If the real intention of this schedule is to provide for pricing work which proves to be of different depth from that described in the bill, then

the Author ventures to suggest that a schedule of percentages representing the increase or decrease required for deeper or shallower sewers than those at anticipated depths is a more suitable provision." Mr. Carpmael found it somewhat difficult to appreciate the difference between a variable schedule of prices to meet variable conditions, and a percentage variation in prices to meet such conditions.

On p. 10 the Author said, " There is a strong body of opinion that a standardized form of contract may not prove satisfactory in every case, and, if the specification is to be regarded as subordinate to the General Conditions on all matters where there is divergence, the Author respectfully agrees." Mr. Carpmael, however, felt very strongly that there should be no divergence whatever between the General and the Special Conditions of Contract. The former should be drawn in such a way as to be comprehensive and applicable to all contracts with which any particular organization was concerned, whilst the latter should be applicable, without divergence from or overlapping of the General Conditions, to particular contracts.

Under the heading " Maintenance and Defects " the Author referred to the condition sometimes placed on a Contractor of making good defects in works for which defects he could have no responsibility whatever. It could hardly be accepted as being equitable to call upon a Contractor to maintain or even to construct a building or to carry out works either inherently defective in design or subject to physical subsidences, both of which were entirely outside his control or responsibility. Under the General Conditions of the Great Western Railway, the Contractor was given the benefit when he could show that the Company or their servants or agents or a third party were in default. The Author said that the Association contract in clause 39 attempted some solution of the problem. It would be interesting to know whether he could suggest a better solution or a more appropriate wording.

With regard to the Author's comments (p. 17) on the provisions relating to " Alterations, Additions, and Omissions," the Great Western Railway General Conditions specified that " Any additional or extra works which the Contractor may be required to carry out shall also be governed so far as applicable by the same terms and conditions and by the same specification," and, in the event of divergence of opinion arising between the Contractor and the Company as to the applicability of that clause and the financial effect of its application, recourse to arbitration was open. With regard to prime cost and provisional sums, the Great Western Railway specified that " It is the duty of the Contractor to see that the supply of all such goods or materials or the carrying out of work as above mentioned shall be governed by the terms and conditions of his contract with the Company." In certain instances the Company themselves obtained quotations for and included as prime-cost items work to be carried out by specialized firms. In such cases it was made clear to those firms that the Company's contract conditions applied.

On the question of arbitration, although for many years he had as Engineer for his Company occupied the invidious position of sole arbitrator, he welcomed the recent introduction into his Company's contract conditions of an arbitration clause. He would point out, however, that such a clause cut both ways. Without it the Engineer would take meticulous care to acquaint himself with all the details, with a resultant settlement favourable and satisfactory to both parties; with it, no matter in whose favour the decision was given, future rancour and unease might originate. In any case, in all railway contracts some matters—for example, the work procedure as affecting traffic working, which involved the safety of the travelling public—had to be left to the unfettered discretion of the Engineer. In general, for the same reason, decisions affecting progress of work (although, as regards Contractors' costs, appropriately referable at a later date to arbitration in the absence of agreement), should be made by the Engineer, by whose decision the Contractor should be bound.

While he most heartily endorsed the Author's opinion as to the necessity for the taking of the greatest possible care to ensure that so far as possible Contractors were made fully aware of the circumstances of any work for which they were asked to tender, he felt that the greatest regard should be paid to "the spirit of the contract"; he ventured to differ in that one respect from Brunel's view and to suggest that if Brunel had not insisted upon the observance of the "letter" as well as the "spirit" of the specification, the settlement of the final account would not have taken 14 years, and heavy legal expenses would thereby have been saved.

In several other ways, which time did not permit him to specify, the Great Western Railway conditions were in accord with the views expressed by the Author.

Mr. M. B. Buxton emphasized that fair conditions of contract were most satisfactory both to the Employer and to the Contractor. They tended to make the job go better, and in almost every case, he believed, they enabled a lower price to be quoted. For many years he had been a member of the Contracts Committee of the Federation of British Industries, some of the members of which were engineers representing various interests. In a semi-judicial manner they had to try to assess whether the terms in various contracts brought up by members were unfair or unreasonable, and it was surprising to find how many terms in the contract conditions of certain municipalities, railway companies, and Government departments, perhaps through an oversight, were in need of revision.

With reference to the Author's remarks under the heading "Alterations, Additions, and Omissions," engineers were aware that variations occurred on every job, and he would like to know whether the Author considered that the power given to the Engineer in most contracts was sufficient, or whether, as fair-minded men, Engineers ought not to ask for rather greater powers than they at present possessed when there was a substantial variation in a contract.

He agreed with the suggestion that the engineering profession demanded standard terms of contract. The Royal Institute of British Architects had had them for 21 years, and their standard terms had recently been revised in a very fair way. Would the Council of The Institution consider setting up a Committee to go into the question and to obtain the benefit of the advice of those with experience in the matter ?

Mr. Hugh Beaver observed that, whilst it was in the interests of all parties to have a fair contract, the interests of the several parties could not be identical. A contractor was employed principally for two reasons: namely, to secure the most experienced person in carrying out the class of work involved, and to relieve the Employer of responsibility. The point to be emphasized in a civil engineering contract was that the Engineer had the dual position of specifying and indicating for the Contractor's assistance how works were to be carried out, and yet had his duty to the Employer in seeing that the responsibility for fulfilling the contract remained always on the Contractor.

If a standard form of contract and General Conditions were to be drawn up, the first stage, so far as engineers were concerned, was to decide what was wanted and what were the ideal conditions. When those matters were decided, it was time to discuss the matter with the opposite side, for, whatever was said, there were bound to be two sides to a contract. A further difficulty was the different degree of responsibility and authority that the Engineer might hold, and the growing interference of the Employer in the administration of a contract.

The Institution should consider the question of standardized conditions of contracts, particularly for structural engineering and building. Most building contractors were accustomed to the R.I.B.A. form of contract, but that was not a good contract; in fact, he had seen a recent legal opinion to the effect that it was not a contract at all.

Standard General Conditions of Contract should make allowance for the different conditions that applied to different classes of contract, such as " cost-plus " or " building-fee " contracts, or the " value-cost " contracts of the London County Council, or the common Continental system where the Engineer priced the bill and the Contractor tendered by percentage, additions, or deductions. There was, for instance, nothing more debatable in a contract than the definition of " cost," and there were numerous problems peculiar to the " cost-plus " contract, such as the question of maintenance and renovation of bad workmanship or poor materials.

Another matter of importance was that of insurance. He was satisfied that the ordinary insurance clauses in the ordinary contract did not give to the owner the protection that was thought. In some cases and circumstances it was fairly certain that there was, in fact, no protection for the Employer at all—especially where there was a large number of contractors on the same job—in regard to third-party risks. He thought that the only

safe way was for the Employer to take out the policy himself, or to lay down his exact terms.

So far there was no insurance for the Engineer. There were policies in existence for architects, but he knew of none for engineers. The Engineer was, however, incurring a definite risk, in spite of all the efforts of the usual contract to place on the Contractor responsibility for the suitability of the ground, for the stability of work during construction, and so forth. In Canada, incidentally, the law put those responsibilities on the Engineer as well as on the Contractor. If, as the Author suggested, the Contractor's degree of responsibility might actually be altered by variations and modifications of the original contracts, plans, and specifications, then the risk might be really serious to both Employer and Engineer.

Another important point was the power of modification, and with it the power of extension or reduction of a contract, or its termination. It had always been Mr. Beaver's firm's view that Engineers—acting either on their own or on behalf of the Employer—should, so far as the Contractor was concerned, have almost unlimited power in that respect. The Contractor was, however, always entitled to claim extra payment, and in the last recourse to have the contract declared void and to be dealt with on a *quantum meruit* basis, should it be so altered as no longer to be that which was reasonably contemplated by both parties at the outset. That view was by no means universal, and at least one important Government department retained, or used to retain, the right to terminate any contract on a month's notice, paying only for the work done. There had also been cases where part of an accepted contract had been cancelled and then placed at a lower price with another contractor.

He would like to ask the Author if an extension of time should be given at the time when the alleged cause or reason for it occurred, or at the end of the contract.

There was an interesting point in regard to constructional plant that was overlooked in the Paper. Almost universally the contract vested the ownership of all temporary plant in the Employer during the period of the contract. What, therefore, was the position when the Contractor did not own the plant—when, as was often the case, he hired it, or was purchasing it by instalments? In Canada and the United States the Contractor generally formed a separate company to own the plant. The security, which it was thought the Employer possessed, was thus often illusory.

There were many other points that would have to be settled when conditions of contract came to be standardized, such as the position and responsibility of sub-contractors and nominated sub-contractors; at the present time they were often sub-contractors only in name, and it was in practice impossible for the Engineer not to deal direct with them. Further, the American practice of sub-letting whole contracts, or the major part of them, was growing in Great Britain. Again, had the Contractor any right

to object to the resident engineer, or to a change of resident engineer? The Author recommended that the term Resident Engineer should be defined; from some points of view it had often seemed wiser to leave that term undefined. Arbitrators were, in fact, already rather inclined to allow the fact that the resident engineer had "acquiesced" in a certain course of action by the Contractor, to influence them in considering whether the Contractor should be made to implement his contract literally. Wisely, they gave no reasons for their decision, but it might be apparent from the award that that had been their view. For that reason it might be best to leave all power only to the Engineers.

There were two questions that he would like to ask the Author. Was it advisable to have an item in the schedule for every clause or sub-clause in the General Conditions that involved, or might involve, expenditure by the Contractor? Why did he not recommend bonuses for early completion?

Finally, Mr. Beaver would suggest that in addition to the General Conditions, there was scope for much standardization in the specifications.

Mr. N. G. Gedye had been much impressed by the fact that the Author, in his introductory remarks, had emphasized the desirability of any condition put into a general set being capable of being fairly enforced and intended to be enforced. He thought that that was the crux of the whole question. It was desirable to eliminate from existing sets of conditions, standardized or otherwise, clauses which could not be fairly enforced, and which the Engineer, and perhaps the Employer also, did not want to enforce, and that the courts could sometimes avoid enforcing because they considered them inequitable.

It was important not to overload the conditions of contract with matter which could be more appropriately included in the specification. If, for instance, in any particular case it was desirable to prescribe or limit the methods to be adopted by the Contractor in the carrying out of the work, it was preferable to deal with such matters by specification, and he believed that that was the sense of the Author's views as set out under the heading "The Specification." A point not commonly covered by the conditions of contract was that mentioned on p. 9. A clause setting out clearly the basis for adjustment of value of preliminary items was certainly required, and such adjustment was not infrequently desirable when the quantities involved in certain sections of the work were either largely increased or largely decreased.

It was very difficult indeed to find a satisfactory solution of the problem of the Contractor's liability for all damage, whether to the works or to adjoining property. Should that include or exclude damage to or failure of work carried out in accordance with the design and specification and resulting from inherent weakness in, or insufficiency of, the designed work? The Association's clause 39 dealt satisfactorily with the case of damage during the period of maintenance, but it did not satisfactorily cover the special case just referred to.

There would seem to be little doubt that variation orders for protective or strengthening works, when the magnitude of the additional work involved was relatively small, should be given by the Engineer without the necessity of first seeking the authority of the Employer, and where a question of safety was involved the Engineer should be empowered in any case to make a variation order. It was perhaps desirable that there should be some clear indication in the conditions of that power and intention, but where was the line to be drawn ? The power could hardly be an unlimited one.

With regard to property in materials and plant, there was one class of plant in connexion with which the usual clause was a potential danger to the Employer. In the case of work in harbours and rivers, where barges, dredgers and so on were employed by the Contractor, claims in connexion with third-party marine risks might fall hardly on the Employer in the event of the Contractor's default in meeting a claim. In such cases a clause was sometimes provided exempting such plant from the usual proviso of property in materials.

The Author's observations on arbitration were of great interest. He would apparently prefer to eliminate from clause 55 of the Association's General Conditions all, or at any rate some part, of the matters reserved to the sole and final decision of the Engineer. Mr. Gedye would not object to some further limitation of the reserved matters, and he hoped that the Author would amplify his view on that point and would say whether or not his opinion was that all matters should be referable to the arbitrator.

Mr. E. J. Buckton said that there were three points in the Paper to which he would refer.

The first point dealt with the extension of time. The Author called particular attention to the provisions in time-extension clauses that extension should be in full satisfaction of all claims, and referred to the question of allowing an arbitrator to revise awards under claims for extension of time.

Most contract forms made the Engineer sole judge of extensions. That was not as dangerous or unfair as it might at first appear, owing to the difference between the effect of delays caused by the Owner and those resulting from outside causes. In the delays caused by the Owner, by the ordering of extras or otherwise, time ceased to be binding if not extended by the extent of the delay, so that if the Engineer, whatever his powers, did not extend time or did not extend it to the extent of the delay, the Owner lost his right to damages. The contract might profess to leave the Engineer judge of the delay, but if the Owner were responsible for the delay, the Engineer would be bound to extend by the actual delay, as otherwise his Employer, being the cause of the delay, lost his rights. If the delay were due to external causes—accidents, storms, abnormal weather-conditions, etc.—those were certainly not matters to be left to arbitration, perhaps 2 or 3 years after the event, when the circumstances would be

more or less forgotten and when persons who could have given evidence might have ceased to be available. The proper time to settle them was when they had just arisen and the circumstances were fresh in mind, and it should not be allowable for any party to raise trouble afterwards by calling into question a decision of the Engineer, which should be binding upon both sides, for it had to be remembered that, if the Engineer's decision regarding extension of time for causes not due to the Owner could be appealed against, it could be appealed against by the Owner as well as by the Contractor.

The second point lay in the vesting of plant and materials in the Employer and re-vesting them in the Contractor. A purely vesting clause would seem to be unfair to the Contractor unless accompanied by a re-vesting clause. The re-vesting clause was, however, a clog on the efficacy of vesting which made it very difficult, in the case of failure of the Contractor, for the building Owner to derive any benefit. Mr. Buckton suggested that a fairer and more practical method was to create a definite charge on the plant, so as to make it answerable in the case of the Contractor failing to fulfil his contract; or, in other words, to effect a deposit of the plant as security for the fulfilment of the contract by the Contractor in the same way and with the same effect as any deposit made as security for the carrying out of a contract.

The third point was that the Author thought clauses 55 and 56 of the General Conditions of Contract of the Association of Consulting Engineers needed amendment. Clause 55 referred to disputes settled by the Engineer, and to the quality of work carried out and materials supplied by the Contractor. It said nothing regarding the quantity of work necessary to fulfil the contract requirements, and the kinds, as distinct from the quality, of the materials to be used in specified works; neither did it deal with the instructions regarding the order in which the work was to be carried out, nor questions relating to the nature, adequacy, and suitability of the Contractor's plant and materials, all of which matters should be decided by the Engineer.

Clause 56 was an "arbitration" clause. The well-known "dispute-prevention clause," although so often associated with the late Mr. A. A. Hudson, was not originated by him, as it had been in use long before his time. It arose from the practice of making the Engineer's certificate final and binding in all matters in connexion with which it was issued, in consequence of which, by the issue of the certificate, all difficulties or disputes were prevented from arising, so that the Engineer in such cases came to be called, about the middle of the last century, a "preventer of disputes," as distinguished from an arbitrator, who could only exercise his functions after disputes had arisen. Years ago, in one of Mr. Buckton's firm's contracts, they had a "dispute-prevention clause" followed by the ordinary "arbitration" clause, and one contractor, seeing that everything was covered by the "dispute-prevention clause," wrote in and asked what

was left for the " arbitration " clause. The reply he received was " everything not covered by the dispute-prevention clause " !

Under a proposed Bill which, about 2 years ago, passed the House of Lords, it was intended to make it illegal for the Engineer to arbitrate. Actually the Engineer was the one man who really knew the subject, and often both parties wished him to handle it ; if, therefore, both parties wished him to settle disputes, he should be allowed to do so. If that Bill became law, in the case of disputes the Engineer would be shut out and Counsel substituted ; the whole atmosphere would be changed and would invariably become false, as descriptions would be given by men who had never been on the works and who were merely describing what had been described to them, at second or third hand. The Engineer was perhaps the only person in a position to give a fair and just decision, but he would be forced into the witness-box as a partisan witness on behalf of the Employer, and he would be treated as adverse by the Contractor's advocate. The natural result of such a law would be to deprive Engineers of any position of independence which they might have attained, and to deprive them of the quasi-judicial functions with which they were at present often entrusted. As, however, the Engineer would always remain the only person who knew most about the work and the circumstances in which it was carried out, it was questionable whether contractors would benefit from the law, for the Engineer's knowledge, instead of being used independently and in fairness to both sides, as he was bound to use it at present, would be driven to be employed on one side only, and for one purpose only, namely to invalidate the Contractor's claims. He would be driven to it as the natural consequence of the impossible position in which he would be put by the procedure, which would result from his being deprived of capacity to give any quasi-judicial binding decision and from his becoming merely a witness on behalf of the owner ; in contracts of real importance the Contractor knew by experience that he had nothing to gain in not having the Engineer as a fair and independent man, holding the balance even between the parties, and very much to lose by having him as an inevitable opponent.

The Author also referred in other places to the Standard Conditions of the Association of Consulting Engineers. They had been published about 8 years ago, but so far Mr. Buckton's firm had been unable to use them. Only last week they reviewed them once more for a particular job, and found them unacceptable. Others might be more happily placed, but usually large civil engineering works were carried out by a schedule contract (namely, measure-and-value principle) and that was not suitably catered for in the Association's standard form.

The great objection to the form in connexion with large civil engineering works was that it was based upon the lump-sum principle, with quantities part of the contract, and was therefore essentially subject to the legal effects and limitations of such forms. It was difficult to see why the Association had not provided two forms, the present one for use in suitable

cases and a "measure-and-value" form for cases where the quantities could only be estimated and where the intention was to pay for the work actually executed as measured.

In the form, contract time was stated to be of the essence of the contract. He believed that it had been held that time could not be essential if, as in the present case, it could be extended or compensated for by liquidated damages. The words, in addition to having no effect, were misleading to laymen.

Under the Standard Conditions the plant was vested with a re-vesting condition which, as he had already said, led to many difficulties. In the only case of that nature with which his firm had to deal, when the Owner, having had to complete the work on the Contractor's failure, attempted to sell the plant on the termination of the work, he was stopped by the Trustee-in-Bankruptcy and the plant was ordered to go to the Trustee and to be sold for the benefit of the general body of creditors. When put to the test, the clause had been found, in the case mentioned, to be valueless.

There was also a number of lesser points, and generally the Association's Standard Conditions, he suggested, were due for an extensive revision.

Standard Conditions, drawn up by collaborating engineers, contractors, quantity surveyors, contract lawyers, and other interested and experienced persons or bodies, could be most useful, but too much faith should not be placed in their ultimate adoption for general use. In any case there should be no attempt to make the form compulsory, and it should be kept flexible enough to have some appeal to the large and important Corporations which had their own views and experience, and rightly did not want to be dictated to in the matter of the conditions under which they were prepared to place contracts for the cost of which they were responsible.

Mr. Buckton's view was that reasonable discretionary powers should, where possible, be allowed to Engineers under a contract. Admittedly that was very difficult and could only be done by express provisions in the conditions, as the Engineer was not a party to the contract and had only such power as he was given. He believed that, under common law, a contractor undertaking to do work at a price fixed or accepted by him, took all the risks of the execution of the work, and whether or not the clause placing all risks upon the Contractor was inserted in the contract, the effect was the same. It did not necessarily follow, however, that the parties could not agree to make some provision for extra payment if unexpected and unforeseen difficulties arose in such matters as, for instance, deep foundations. Whatever was done, however, it should be remembered that an Engineer had no more power than an arbitrator or a Judge to make after the event a contract for the parties other than that which they had made for themselves.

Mr. Bryant Irvine said that the Author had given a very interesting history of the evolution of the present arbitration clause, but that he did not answer a question which had been in Mr. Irvine's mind for some time;

namely, why the clause as at present drafted had such great and universal popularity. Mr. Halcrow mentioned one of the limitations to be found in that clause. Section 56 of the Association contract said that the award of the arbitrator " shall be final and binding on the parties," and Mr. Halcrow remarked that it would be of great advantage to members of the profession if those words could be taken to mean what they said. There were, unfortunately, two very important difficulties in the way which could not be overcome.

The first difficulty was contained in the Arbitration Acts. Section 7 of the Act of 1889 provided that arbitrators should have among their powers the right to send any award which they made in the form of a special case to have the opinion of the Court ; and it was further provided in section 19 of that Act that the arbitrator might, at any stage of the proceedings, state his award in the form of a special case for the opinion of the Court on any question of law arising in the course of the reference. In the last few years similar provisions had been made by Section 9 (1) of the Arbitration Act of 1934. During the evolution of the present arbitration clause, therefore, that possibility had always had to be considered.

The second difficulty was the general legal position which had arisen from the decisions in cases where people had endeavoured to get over the provisions of the Arbitration Acts. There was an example of that in the case of Czarnikow *v.* Roth, Schmidt and Company [1]. A clause had been inserted in the contract in that case which provided that no person " shall require, nor shall they apply to the Court to require, any arbitrators to state in the form of a special case for the opinion of the Court any question of law arising in the reference, but such question of law shall be determined in the arbitration in manner herein directed." That was exactly what Mr. Halcrow said would be a great advantage, if it were a clause which was binding on the parties. Scrutton, L.J., said in that case [2] :

" Arbitrators must understand that parties before them have a right to take the opinion of the Court as to whether the arbitrators should be given the guidance of the Court in matters of law, and that they must not attempt to stop the action of the Courts by interfering with or hindering such a right of parties."

He also said [3] :

" The Courts . . . do not allow the agreement of private parties to oust the jurisdiction of the King's Courts. Arbitrators, unless expressly otherwise authorized, have to apply the laws of England. When they are persons untrained in law, and especially when as in this case they allow persons trained in law to address them on legal points, there is every probability of their going wrong, and for that reason Parliament has provided in the Arbitration Act that, not only may they ask the Courts for guidance and the solution of their legal problems in special cases stated at their own instance,

[1] 1922, 2 K.B., 478.
[2] *Loc. cit., p.* 490.
[3] *Loc. cit., p.* 488.

but that the Courts may require them, even if unwilling, to state cases for the opinion of the Court on the application of a party to the arbitration if the Courts think it proper."

For those two reasons, it was impossible for the words stating that the award of the arbitrator " shall be final and binding " to mean what they said.

Over a period of years Mr. Irvine had asked his friends in the engineering profession whether they could explain to him the real reason for the great popularity of the arbitration clause, and it was in the hope that the Author would be able to provide him with a solution to that problem that he had referred to the matter. Realizing those two difficulties, it was easier to examine the four matters which were usually raised by members of the engineering profession in defence of the arbitration clause.

The first point was that arbitration had the advantage of speed. When an action was brought in the High Court there was the possibility of an appeal to the Court of Appeal and then to the House of Lords. When difficulties were submitted to arbitration and the award was not accepted, the case stated by the Arbitrator came before that Court in which an action started in the High Court received its first hearing. There was then still the same possibility of appeal to two further Courts. That being the case, it was clear that the arbitration clause introduced the possibility of four hearings, whereas if the difficulties were submitted to the High Court in the first instance, there would only be the possibility of three. For that reason the arbitration clause was bound to mean that the final determination of the difficulties might only be achieved by a longer route.

Further, if it were suggested that it took some time to get heard in the High Court, it would certainly not be quicker to obtain the award of an arbitrator and then have to take a place, if there were an appeal, in the list in the High Court in which the action could have been set down many weeks earlier had there been no arbitration.

Further, with regard to the actual time taken up by the hearing, it could hardly be claimed that that was inevitably shortened where the proceedings were conducted by an arbitrator. Unless the arbitrator was one who spent most of his time on the work, it was unlikely that he would have the grip of the principles of evidence and the appreciation of the quickest method of reaching the point which those who appeared before the High Court expected as a matter of course from a Judge.

The second suggestion that Mr. Irvine had heard made was that it was cheaper to go to arbitration. That did not appear to bear careful scrutiny. In the first place solicitors and Counsel were employed in any important arbitration in exactly the same way as if the matter were coming before the High Court. There was also the possible expense of an additional hearing, as the proceedings were starting by arbitration and not in the High Court. Finally, the fees which were paid to the arbitrator and for

the hire of a hall were bound to compare very unfavourably with the actual fees paid for a hearing in the High Court.

The third suggestion was that by arbitration the parties avoided the mysteries and expense which were involved in obtaining the services of lawyers. From what was set out above it would be clear that that suggestion had no substance whatever, desirable as it might be if it could be achieved.

Lastly, it was suggested that the advantage of arbitration was that engineers were able to submit their disputes to an engineer for determination. If that were the real reason, Mr. Irvine was in favour of its being made clear, and of abandoning the other explanations which were so frequently made. If a speedy determination of a dispute were required, and it was felt that an engineer was better fitted to achieve that result than a Judge, even though the latter was generally fully experienced in obtaining the assistance of expert technical evidence and quickly assessing its value, possibly the time had come when The Institution might consider the appointment of a small panel of arbitrators who were qualified both in engineering and in the law. The Tribunal of Appeal appointed under the London Building Act of 1930, and the London Court of Arbitration, provided examples of arbitration under the best conditions. In those cases the arbitrator was usually a man with special knowledge of the problem before him, who at the same time was fully conversant with the legal difficulties which might arise. If only the first of those qualifications were demanded, as appeared to be the case in some arbitrations, the final award had to run the risk of being obtained only by a longer and more costly route than would otherwise have been the case. Unless both those qualifications were found in the Arbitrator, the final determination of the dispute would in many cases be obtained more quickly and cheaply if there were no arbitration clause in the contract, and the dispute were submitted at once in the ordinary way to the High Court of Justice.

Mr. W. S. Kennedy thought that there were two basic principles that ought to exist in contract conditions. One was that the Engineer should have entire power over the work: the Engineer should be allowed to say what the work had to be done in a particular manner, if he considered it better and safer, and he should be allowed to reject materials which he considered unsuitable. The second was that the Contractor should be allowed to put in a claim to an impartial arbitrator at the end of the contract if he thought that any of the Engineer's orders had occasioned additional expense.

If those principles were applied to the power to order variations, it was necessary to make a certain exception. The Author stated on p. 6 that "... no Court interpreting the contract will readily disregard the fact that the contract was entered into on the basis that the works as shown on the contract drawings were substantially those which the parties expected to

be carried out and (unless expressly stated to the contrary) were the works for which the Contractor made his tender, calculated his on-costs and generally arrived at his contract prices." The Author mentioned drawings, and possibly he would extend that to include quantities. It would appear, however, that there was bound to be some limit on the power of the Engineer to order variations of quantities, and Mr. Kennedy therefore considered that a percentage variation (perhaps 10 per cent.) would be satisfactory. The Author, however, objected to percentage limitations altogether. It was well known that hard cases made bad law, but one of the first big arbitrations in which he had been engaged [1] dealt with the construction of the railway from Wootton Bassett to Patchway. It was the case of a measure-and-value contract. The Engineer had specified certain quantities which were to be put into an embankment, but the material would not stand at the angle which both parties had assumed, with the result that the amount of material which had to be put into the embankment was more than doubled. The Contractor's view—with which Mr. Kennedy agreed, although he did not say so at the time!—was that if the quantity of material were doubled, the work would be altogether outside the contract, as a contract for 2 million cubic yards was not the same as one for 1 million cubic yards; the extra material could not be obtained from the cuttings and he would therefore have to fetch it from elsewhere, entailing a great deal of extra expense. The Contractor accordingly asked for a larger cost per cubic yard than the original measured-value price. The Great Western Railway naturally held the view that the contractor tendered at 1*s.* 6*d.* per cubic yard, and that as the contract stated the quantity to be "more or less" 1 million cubic yards, the fact that there was more should make no difference to the price. The litigation lasted 4 years and Mr. Kennedy thought that it would have been very much more economical if the parties had understood that that kind of variation was not within the terms of the contract.

The Author mentioned the relationship of the Contractor with the Engineer and the Employer. Mr. Kennedy considered that the Contractor's position was a relation with the Engineer as the Employer's agent, and with the Engineer alone; he did not think that the possibility should be considered of the Contractor making an appeal to the Employer because he thought that the Engineer had done something which was not right. The Employer and the Engineer should consult together, and the Engineer should tell the Contractor the decision of the Employer and the Engineer.

The President said that several speakers had referred to the desirability of revising the General Conditions of Contract in order to come into line with modern practice. The Institution had decided to meet the Federation of Civil Engineering Contractors, so that the existing terms of General

[1] Pearson *v.* The Great Western Railway.

Conditions of Contract could be considered with a view to standardizing them as far as possible.

*** **Mr. F. M. G. Du-Plat-Taylor** pointed out that there was a tendency to reduce General Conditions of Contract to a common formula, but that the form of those conditions had to vary with the class of work and risk.

In sea-defence work the risks were greater than in any other class of work, as the risk of damage by wind and waves, either during the execution of the work or during the period of maintenance, was almost impossible to assess. He had, for instance, known a foreshore, which had remained in a more or less stable condition for many years, to be washed out by a series of onshore gales so that its level was reduced by over 10 feet. Such an occurrence would undoubtedly have proved fatal to any structure designed on the basis of the condition of the foreshore as previously known.

Further, the stability of a sea-wall might be contingent on the proper upkeep of other structures, such as groynes. That was the case in JACKSON *v.* THE EASTBOURNE LOCAL BOARD (1884), a suit which was carried to the House of Lords. The Contractor was held to be responsible for the failure of a sea-wall, although that was admittedly mainly caused by the failure of the Local Board to maintain or to repair existing groynes, or to provide additional ones. Such a risk was one which, if thrown upon the Contractor, would be bound to result in very high prices, and in those circumstances it was one which Mr. Du-Plat-Taylor generally advised his clients to take upon themselves.

Schedules of prices were mentioned on p. 7. Such schedules should in his view form part of every contract. They were required, in any case, for assessing the value of work carried out as day-work, and should therefore include for such things as the use of plant, and the use and waste of timber and other materials required for temporary works.

On the question of extent of work, mentioned on p. 11, he suggested that it was, in many cases, advantageous to include in the Conditions a provision that the prices stated in the bill of quantities should hold good for quantities up to 25 per cent. in excess of, or 25 per cent. less than, the quantities shown. That might give the Contractor some benefit in case of an excess quantity being ordered, but it cut both ways.

He suggested that the words "substantial completion" were the appropriate ones to use in a certificate of completion, as better expressing the state in which a certificate could be issued, where various minor matters still remained to be finished.

With regard to the question of the seizure of plant on discharging a contractor, the plant was really held as a security for the original contractor meeting his obligations under the contract: that was to say, for the excess

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*** This and the succeeding contributions were submitted in writing.—SEC. INST. C.E.

cost which might be incurred by completing the work by other contractors and for liquidated damages, if any. In one well-known case a contractor was turned off a very large job and his plant was seized. A law-suit followed and the plant remained on the site until the suit had been finally decided by the Court of Appeal. By that time the works had been completed by another contractor, and the original contractor's plant had become both rusty and obsolete, and was practically worthless. The second contractor made no use of it. It seemed to Mr. Du-Plat-Taylor that if the impounded plant were efficient and in good condition, the proper course was for the substituted contractor to use it, and to allow the building owner reasonable hire rates for it. In the final settlement with the original contractor, the sum so obtained should be put into hotchpot, and any balance remaining after the extra cost of the works and all claims against the original contractor had been met, should be paid over to the latter. It would be necessary for the protection of all parties that the impounded plant should be surveyed by a competent mechanical engineer both before and after its use by the second contractor, so that any damage that the plant might have sustained whilst in use by the latter should be paid for by him.

Sir Lynden Macassey observed that the complexity of a public-works contract could readily be appreciated by comparing the form of contract of to-day with the simple form in use 75–80 or more years ago. In those days, railways, harbours, docks, bridges, waterworks, etc., were being constructed in profusion, and design and construction were certainly, with the methods and materials then available, as difficult as they were to-day. It might, then, be fairly asked, how had the present complexity been developed ? The answer, he thought, was that in the main it was due to the attempts of engineers, under pressure from, or with the too-willing assistance of, lawyers, to try to foresee, and to insert provisions to cover, every kind of circumstance and condition which might possibly arise, or contingency which might be encountered, in carrying out the works, followed by subsequent modifications of those provisions to try to come within, or to keep without, decisions in the Courts.

Whether that had been in the true interests either of the engineering profession or of their clients was very doubtful. In the old days a company or authority carrying out public works was prepared without hesitation to leave decisions on all matters of subsidiary detail to the Engineer, in whom it invariably placed full professional confidence. That was obvious from the striking simplicity of the old-time public-works contract. Might not the professional status of an engineer and his authority over his client be higher to-day if so many refinements in its provisions had not been introduced into the form of contract, and the subject of such refinements had been left as was originally done, to be treated as ordinary matters incidental to the carrying out of public works which the Engineer himself would decide and determine if and when they arose ? It might reasonably be conjectured whether that would not also have inured to the material

benefit of the engineer's client in the shape of lower-priced tenders. To-day, however, the public-works contract in all its complexity had become so largely a matter of common form in Great Britain that no doubt to simplify it would be difficult. The only reform probably now practicable in Great Britain was the standardization in a reasonable form of the provisions of all such contracts. Such standardization would be the next best thing to simplicity if the latter were unattainable.

Since the complexity of a modern English public-works contract would probably continue in Great Britain, if indeed it did not further develop, it was worth while, in Sir Lynden's view, to consider it from a wider standpoint. The form of contract in use in foreign countries, and indeed in most of the Dominions, was in many respects different and altogether simpler. Those who had had to negotiate concessions with foreign governments, or contracts with authorities or companies in foreign countries, for the carrying out of public works by English construction syndicates or contractors, knew the ineradicable suspicion which the complexity of the present-day form of English public-works contract always aroused. Indeed, sometimes that suspicion was so strong that it was frequently quite impossible to secure the adoption of the English form, and another and simpler form of contract had in the end to be accepted. Having regard to the comparative simplicity of the form of contract which foreign interests were accustomed to, and preferred, that had for some time raised in Sir Lynden's mind the question whether or not there was any sound reason for English engineers and contractors, who were nowadays handicapped by so many competitive and other disadvantages in securing contracts for, and in the carrying out of, public works in foreign countries, having their difficulties increased by the use of a nationalistic form of contract to which there was such general international objection.

There had been no more remarkable development in the commercial world than the way in which great English international trading and commercial organizations, which at one time were able to insist upon a characteristically English form of contract, had now abandoned it, and had adopted a form which had been agreed with foreign organizations allied in business with them. Many English engineering firms, when they were supplying machinery in Great Britain, would offer or accept a form of contract in common use amongst English mechanical engineers which did not differ substantially in its main provisions from the common form of English public-works contract, yet if they were offering to contract for the supply of the same machinery in a foreign country, they would never think of doing so on that form of contract, but would proffer a form which was in common international use for the supply of machinery. They knew that to quote on the basis of the complex standard English form would probably involve the rejection of their quotation.

There was scarcely any great international trade centred in Great Britain in which an international form of contract had not now been introduced.

The usual way in which it had been done was by the institution by the English trade organization of consultations with foreign organizations representing interested parties. In that way a form had been ultimately evolved which commanded general approval, and which before very long by common consent and practice became the accepted international form.

To give an illustration, at one time the provisions inserted in the Bills of Lading and Charter Parties in use in different countries of the world regarding general average were of an extremely diverse and conflicting character. The International Law Association, at the instance of certain great commercial interests, instituted an inquiry through its various foreign branches, and eventually, after several international conferences, succeeded in formulating "Rules of General Average." After subsequent amendments which experience had showed were desirable, the rules so framed came to be adopted by general agreement all over the world, with the greatest advantage to international trade, as the York-Antwerp Rules of General Average, the names being derived from the places where the conferences were held.

Another illustration of the prevailing tendency was afforded by the present proceedings of an international committee, of which Lord Macmillan, P.C., G.C.V.O., K.C., Hon. M. Inst. C.E., was the Chairman and Sir Lynden was the Convener, that was engaged, with the co-operation of important international commercial interests, in trying to formulate a commercial arbitration clause and rules to govern arbitrations taking place under the clause which would be internationally acceptable and could be adopted in any international commercial contract. The representative international trading and commercial organizations all had their own distinctive standard forms of contract available for their members, containing their own special arbitration clause and rules for arbitration. The well-known international courts of arbitration also each had their own arbitration clause available for insertion in contracts to provide for arbitration by the court selected, of disputes under the court's rules. There was, however, a great and growing number of commercial firms and business men, who were neither members of trading organizations nor were desirous of referring their disputes to any of the international courts of arbitration. They were in the habit of inserting all kinds of arbitration clauses in their commercial contracts, and there was frequently the greatest confusion as to what national law or what procedure to apply to the arbitration. That had led the International Law Association to constitute the Committee mentioned above, which was representative of the principal commercial countries, in order to formulate a standard arbitration clause and code of arbitration rules for voluntary adoption in international commercial contracts. There was little doubt that when the clause and rules were authoritatively formulated and promulgated they would soon come to be adopted internationally, just as the York-Antwerp Rules of General Average had been adopted. Many other similar illustrations could be given.

Sir Lynden therefore ventured to submit that The Institution should consider whether it would not assist English engineers and contractors if it followed the same line of progress as the commercial world, and endeavoured to evolve a form of public-works contract for use outside Great Britain, proceeding on lines familiar to, and acceptable to, foreign interests, and at the same time providing effectively for all the main matters for which, according to English practice, it was felt provision should be made. It could hardly be contended, he thought, that the value of the present English form of public-works contract, with all its complexity as compared with the customary simpler foreign form, was worth the detriment to English engineers and contractors which experience showed resulted from attempts to use it for the construction of public works in foreign countries.

Mr. H. F. Payne suggested that either a standard set of General Conditions subordinate in particular scope to the Specification, as suggested by the Author, or a set of recommended clauses with alternatives for use according to the magnitude, complexity or period of the contract, would be very useful.

If a standard set of conditions were adopted, at least two forms, one for small lump-sum contracts accompanied or otherwise by priced bills of quantities or other details of tender sum, and one for measure-and-value contracts, would be required. A separate form of conditions for measure-and-value contracts might also with advantage be provided where the provisional sum sub-contracts were of considerable extent.

The clauses dealing with labour, referred to on p. 14, could better be included in the Specification, save for the general statement that the Contractor was to provide all labour required.

In addition to the methods of valuation of variations given on p. 18, valuation by day-work should be included at the discretion of the Engineer, and the percentage addition to be allowed for the Contractor's overhead charges and profit should be stated.

Where the provisional sum sub-contracts were of such size or nature that they would have to be paid in instalments in interim certificates, the general conditions of the sub-contract should either be to some known standard or should be prepared and inserted *in extenso* in the General Conditions. They should make provision for the direct payment to sub-contractors and for the allowance of a cash discount to the General Contractor.

Mr. C. P. Taylor observed that on p. 17, the Author referred to certificates on "practical completion." Mr. Taylor had for many years introduced in contracts the following phrase :—

"Provided that the Engineer shall not be entitled to reject any part of the contract work which fails to pass any test if he is satisfied that such failure is due only to some minor omission or defect which the Contractor undertakes to make good at the earliest moment convenient to the Purchasers."

Whilst that had been more generally employed in relation to machinery the spirit of it could clearly be expressed in a clause suitable for contracts for constructional work.

On p. 19, under the heading "Property in Materials and Plant," the Author referred to the practice of marking such equipment with the Employers' name, and he used the phrase "in the case of the Contractor not being a Company." Why was that necessary? As a very large proportion of contract work in Great Britain was carried out by Companies, the practice could have very little value unless it could be applied under those conditions.

The Author, in reply, pointed out that the crucial test of the conditions of engineering conditions only arose when there was a difference of opinion between any of the three parties concerned—the Employer, the Engineer, and the Contractor. If the Engineer possessed the complete confidence of the Employer, and the Employer was prepared to leave all matters of difference to be settled by the Engineer, then the Engineer might, as suggested by Mr. Carpmael, interpret the conditions in "the spirit" rather than by "the letter" of the agreement. It had, however, to be borne in mind that Employers were entitled to require the contract to be administered and interpreted strictly, and that therefore the test as to whether or not conditions of contract were reasonable had to be made on the basis of their ultimate legal interpretation. For that reason, if Engineers thought that they should have a discretion to vary rates, to order protective work at the cost of the Employer, to limit liability of the Contractor in certain exceptional contingencies, etc., they had to see that their power to do so was contained in the contract.

The advantages of the Engineer disposing of matters in dispute during the progress of the work were obvious, and the fact that conditions of contract entitled either party to require that the matter should come before an independent arbitrator did not, as suggested by one of the speakers, prevent the parties agreeing at the time of the dispute to waive that right, and to ask the Engineer there and then to give his decision and to be bound by it.

The Author thought that the discussion had emphasized the vagueness of existing forms of contract in regard to the effect either of incorrect bills of quantities or of variations on the Contractor's preliminary items, contingency items, overheads, etc. Moreover, the fact that variations might entirely change the conditions of the work and the Contractor's liabilities, was not, in his opinion, sufficiently provided for in the usual forms of contract.

Clause 39 of the Association contract had limited the Contractor's liability for damage during the maintenance period, but that did not, in the Author's opinion, really touch the question whether or not the Contractor should take the responsibility for failure under load of work which he had carried out strictly to the Engineer's design.

The Author appreciated that the Association clause in its form was one which was based upon a lump-sum called the "contract price," but having regard to the fact that by clause 52 it provided for the modifications of that sum in accordance with the actual measurements of the work, he would have expected that those desiring to contract on the measure-and-value principle would have found it acceptable.

The provisions for vesting of plant were designed for the purpose of additional security to the Employer and—what was often equally important—to ensure that upon the dismissal of a Contractor the Employer might use the plant and materials for completing the work. If there were a vesting clause the Trustee-in-Bankruptcy could only claim the plant if the Contractor were an individual and not a Limited Company, and if there were no indication by the marking of the plant or otherwise that the plant did not belong to the bankrupt.

A Contractor could not, of course, by his contract with the Employer give ownership to the Employer of plant which did not belong to him, and the true owner might take away such plant at any time. For that reason it might be desirable to require that all plant brought upon the job should be the property of the Contractor.

The extension of time for completion should never be deferred until the date of completion had passed or until the end of the contract. It was only right that the Contractor should know when the event which justified an extension took place what additional time he had to complete the work. An agreement to pay a bonus for early completion, in the Author's opinion, was not as a rule desirable, because it cast upon Employer and Engineer the obligations of giving the facilities for completion at an earlier time than the contract time, upon which no doubt they had made their original programme for delivery of sites, plans, etc. The clause was one which naturally often caused differences and disputes. Criticism of the R.I.B.A. form of building contract had been made. It was because that form of contract was dealt with not as a compromise of principle but as a compromise of words that that contract, as had been said, failed to express clearly what the parties intended. A committee representing different interests and compromising on words could make nonsense of a clause; a committee compromising on principles and then handing their decisions to be embodied in one document could fairly expect to have their compromise expressed in clear words. In the contract which he trusted engineers would ultimately have there would, he hoped, be no compromise on the words which were necessary to express the decisions which the parties had previously made.

In that connexion it should be mentioned that the R.I.B.A. form of contract was not the only one in which the contractor's view had been considered and in many respects embodied. That had been done also in the Institution of Electrical Engineers Model Form of Contract for the supply of plant and materials and the execution of work connected there-

with, and there was no doubt that the civil engineering conditions of contract, in the varied forms used to-day, lagged far behind any of the conditions of contract used by the kindred industries which had to make provision for contingencies which might arise in the carrying out of work. It was interesting to note, in relation to Sir Lynden Macassey's comments, that the I.E.E. Model Form of Contract was on sale in New York as well as in London.

*** The Correspondence on the foregoing Paper will be published in the Institution Journal for October 1939.—SEC. INST. C.E.

JOINT MEETING WITH THE BRITISH SECTION, SOCIÉTÉ DES INGÉNIEURS CIVILS DE FRANCE, AND THE INSTITUTION OF STRUCTURAL ENGINEERS.

17 January, 1939.

WILLIAM JAMES EAMES BINNIE, M.A., President Inst. C.E., in the Chair, supported by

WILLIAM THOMSON HALCROW, President, British Section, Société des Ingénieurs Civils de France, and

LIEUTENANT-COLONEL HENRY SCHOFIELD ROGERS, C.M.G., D.S.O., President Inst. Struct. E.

"Strengthening of the Austerlitz Bridge (Paris) by Electric-Arc Welding."

By MONSIEUR FAUCONNIER.

Abstract.[1]

THE Austerlitz bridge, which is a steel three-hinged arch bridge, with a span of 140 metres, was designed in 1903 to carry trains of the Paris Metropolitan Railway, weighing 121 tons, across the river Seine. As the weight of the trains has now increased to 420 tons, with a considerable increase in length, various methods of strengthening the bridge were considered, as well as its re-building, but it was finally decided to use the following method of reinforcement :—

(1) To increase by up to 60 per cent. the working section of all weak members in the structure, by means of plates, etc., fastened by electric-arc welding.
(2) To restore the stability of the lower extensions by fixing, under the bottom flange of each unit, a framework resting on a special bearing.
(3) To reinforce the abutments by building a reinforced-concrete block on the land side, and on the river side a reinforced-concrete sill supporting the new abutment bearings.

Electric-arc welding was selected as it would have been difficult, if not impossible, to apply the riveting process to most of the parts to be reinforced without seriously upsetting their equilibrium, especially as the work had to be carried out while the bridge was carrying its usual traffic.

Considerable thought was given to the selection of the steel to be used for reinforcements. At first a high-strength steel of the Ac. 54 type was favoured, but after taking into consideration the factors of weldability, ductility, and lack of brittleness, it was finally decided to use an extra mild steel, especially as the flow properties of mild steel are able to absorb any

[1] Published in full in Journal Inst. Struct. E., vol. xvii (new series) (1938), 61 (January, 1939).—SEC. INST. C.E.

stresses caused by contraction due to rapid cooling of the weld. Th chemical analysis of the steel used is given in the Paper. For the sake homogeneity the electrode chosen for the welding, one with a semi-volati coating, was such that it was possible to deposit a steel, the physic properties—and especially the ductility—of which would come very clo to those of the materials to be united.

The lower and upper arch-members were reinforced by adding recta gular bars measuring from 70 by 100 millimetres to 70 by 140 millimetre arranged between the rivet-rows and welded on edge, the existing an added parts being carefully machined. The additional frameworks f the lower extensions were constructed on the banks of the river at th foot of the bridge, and during hoisting every precaution was taken t avoid distortion of the parts. The longitudinal beams and cross ties i the bridge-deck were strengthened by adding flanges to their lower portion and by welding their top portions to the deck-plates, the latter being mad to act as flanges. Electric power for welding was supplied in the form o 550–600-volt direct current. It was then converted into 60-volt direc current for general use. Each welder had at his disposal a resistance-bo connected to the 60-volt cables, and could adjust the current up to 22 amperes.

In order to ensure sound finished welds, electrodes were only accepte after rigid tests; and, with regard to the operators, no welder was accepte until he had passed a conclusive test carried out under actual workin conditions and repeated for all members of the welding staff every months. Whenever welds appeared to be defective, and at regular interval in sound-looking welds, soundings were made by means of the Schmuckle milling device, which supplied positive information regarding the quality of the welds. In all, 250 tons of steel, or more than one-third of the origina weight of the bridge, were welded on to the structure.

The Paper concludes with an Appendix dealing with

(1) micro-examination of a butt-joint weld under actual working conditions met with during the bridge-reinforcement operations;
(2) residual stress-relief under external loads; and
(3) endurance of the bridge under repeated stresses after reinforcement.

A vote of thanks to the Author for his Paper was proposed by Mr W. J. E. Binnie, President Inst. C.E., seconded by Mr. W. T. Halcrow President, British Section, Société des Ingénieurs Civils de France, an supported by Lieutenant-Colonel H. S. Rogers, President Inst. Struct. E.

The Meeting concluded with a discussion, in which seven speakers too part. A report of the discussion will appear in the March Journal Inst Struct. E.

Paper No. 5179.

"The Properties of Composite Beams, consisting of Steel Joists Encased in Concrete, under Direct and Sustained Loading."

By Professor Cyril Batho, D.Sc., M. Inst. C.E., Stanley Dale Lash, Ph.D., Assoc. M. Inst. C.E., and Reginald Herbert Honley Kirkham, B.Sc., Stud. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*) [1]

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Introduction.

TRUCTURAL steel encased in concrete is used in many forms of con-ruction. One of the first types to be developed was the filler-joist floor, ut the method is now frequently employed for important structures such multi-storey buildings and reservoirs, and for strengthening steel idges, the steel often being augmented by the addition of reinforcing ds.

[1] Correspondence on this Paper can be accepted until the 15th May, 1939, and will published in the Institution Journal in October 1939.—Sec. Inst. C.E.

Concrete is also extensively used for the fire-proofing of steelwork and as a protection from corrosion, but it is common practice to neglect the strengthening effect of the encasement, although some allowance for it has recently been recommended in the Final Report of the Steel Structures Research Committee [1].

Many experiments have been carried out, mainly in other countries, on composite beams, filler-joist floors and similar structural elements, and tests on filler-joist floors were also made at the National Physical Laboratory in 1922. A brief description of some of the tests is included in a Paper by W. B. Scott [2], and a summary of the chief conclusions reached, together with a method of design for composite beams, will be found in a recently published book on reinforced concrete by R. A. Caughey [3]. Amongst other publications a Paper by L. Baes [4] describing tests on T-beams and floors may be mentioned. Investigations on the effect of the concrete encasement on the connexions of steel frames are described in the Final Report of the Steel Structures Research Committee [5].

Most of the investigators reached the conclusion that, if the bond between the joist and the concrete is sufficient to ensure composite action, the stresses in composite beams may be estimated by the methods used for ordinary reinforced concrete. In reinforced-concrete beams the ends of the rods are usually hooked in order to anchor them to the concrete. In a composite beam the joist will continue to take part of the load if the bond fails, but Baes and Caughey both advocate the addition of anchorage angles to guard against the effect of such deficiency.

The investigations described in the present Paper were commenced some years ago, and were carried out in the Civil Engineering Laboratory of the University of Birmingham. They were initiated because it was considered desirable to make as thorough an examination as possible of the fundamental properties of composite beams. In order to limit the number of factors, it was decided, apart from subsidiary tests, to confine the investigations to simple beams consisting of a steel joist, with or without anchorages or end connexions, encased or partially encased in concrete. In the first part of the Paper experiments carried out directly to failure are described, and the second part deals with experiments under sustained loading, including investigations on the effect of exposure to open-air conditions, a subject on which there appears to be little information available either for composite or reinforced-concrete beams. The effect of

[1] Department of Scientific and Industrial Research. H.M. Stationery Office, 1936.

[2] W. B. Scott, "Augmented Steel." Journal Inst.Struct.E., vol. 6 (1928), p. 344.

[3] R. A. Caughey, "Reinforced Concrete." London, 1937.

[4] L. Baes, "Poutrelles métalliques enrobées." *L'Ossature Métallique*. January-February, 1933.

[5] C. Batho and S. D. Lash, "Further Investigations on Beam and Stanchion Connexions." Part 2, p. 328. H.M. Stationery Office, 1936.

creep and shrinkage on the load-factors on yield of the steel [1] and ultimate strength [2] is also considered.

When experiments are planned, and during the course of an investigation, it is not always evident what information will ultimately prove to be of value, and in dealing with a material like concrete, particularly under sustained loading, the filling of gaps entails considerable expenditure of time and materials. There are gaps in the evidence obtained in these tests. For example, the desirability of measuring the widths of cracks was not realized until it was too late. More information is also desirable on the effects of anchorages, but it is hoped that sufficient evidence has been obtained to give a clear picture of the behaviour of composite beams.

It is necessary to be able to distinguish briefly between tests carried on directly to a finish and those in which the loading is maintained for a long period. These will be referred to as tests under "direct loading" and "sustained loading" respectively.

Fully and partially encased joists are often referred to in the text as fully encased or partially encased beams in order to avoid confusion in the context.

Summaries of the results obtained and the conclusions reached will be found at the ends of Parts I and II.

PART I.—EXPERIMENTS UNDER DIRECT LOADING.

Description of the Specimens and Method of Testing.

Particulars of the specimens are given in Figs. 1, Plate 1. These included two series of composite beams Nos. 1 to 7 and 8 to 12, and two beams, Nos. 13 and 14, having flat plates as reinforcement. A reinforced-concrete beam, No. 15, having horizontal tension bars and vertical stirrups was also tested for purposes of comparison.

The same method of loading was used for all the specimens, the span being 6 feet 8 inches and the loading points 2 feet 2 inches from each support. Equal loads were applied at the two loading points.

Specimens Nos. 1 to 4 of the first series of composite beams were similar, but the last two were loaded and supported directly on the steel as shown in Figs. 1, Plate 1. One of the partially encased specimens (No. 6), had angles welded to the encased flange of the joist, and in another (No. 7), the top of the encased flange was covered with oiled paper in order to destroy the bond.

The specimens of the second series were made with a stronger mix of concrete and a greater cover over the compression flange of the joist.

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[1] $\frac{\text{Load at yield}}{\text{Working load}}$.

[2] $\frac{\text{Load at failure}}{\text{Working load}}$.

Otherwise Nos. 8 and 9 were similar to the beams in the first series. In beam No. 10, the joist was split into two separate parts by a cut in the web under the compression flange parallel to the axis. The object of this was to examine the effect of preventing direct beam-action in the joist. Specimen No. 11 had angles bolted to the top flange, and compression reinforcing rods were added in specimen No. 12 in order to bring the neutral axis above the joist. The two beams with flat plates were used to compare the bond on vertical and horizontal plates.

All the specimens of this series, including the beams with flat plates as reinforcement and the reinforced-concrete beam, were proportioned so that the calculated maximum tensile stresses in the steel at a given load were approximately the same.

The steel plates and reinforcing rods were of mild steel of ordinary structural steel quality, and were not machined or surface-treated in any manner before use. Care was taken to obtain rolled steel joists as true to normal shape as possible.

Two mixes of concrete were employed :—

(*a*) For beams Nos. 1 to 7, the mix was 1 : 2 : 4 by volume, using rapid-hardening Portland cement, Leighton Buzzard sand, and a washed gravel aggregate graded from $\frac{3}{4}$ inch downwards. The water/cement ratio by volume was 0·74 (allowing for absorption). The compressive strength of 6-inch cubes at 28 days averaged 3,100 lb. per square inch.

(*b*) For beams Nos. 8 to 15, the mix was 1 : 1·5 : 2·5 by volume, the same materials being used with a water/cement ratio of 0·59. The compressive strength of 6-inch cubes at 28 days averaged 5,500 lb. per square inch.

In both cases the actual proportioning of the materials was done by weight to obtain greater accuracy, and the mixing was carried out in an open-pan machine mixer for 2 minutes. The beams were kept damp for 10–12 days after casting.

The beams were tested about 28 days after pouring, and were loaded and supported in the manner shown in *Fig. 2*. They were covered with a thin wash of white " Ceilingite " in order to facilitate the observation of the development of the cracks.

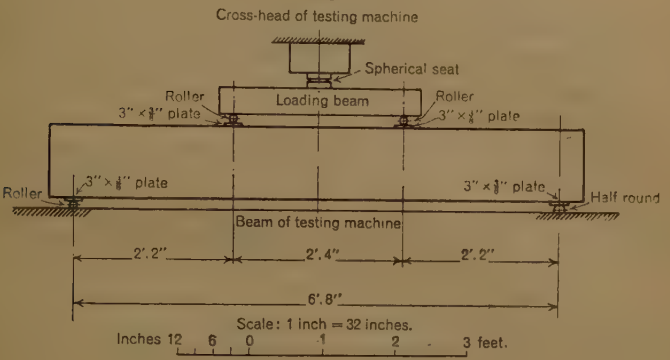
Strain measurements were taken on the surface of the concrete in the middle portion of the beam by means of mirror-extensometers having a gauge-length of 8 inches. Readings were taken on both sides of the beam in order to obtain the average strains and also to eliminate errors due to any movements of the beam as a whole. In certain of the tests strains were also measured on the top and bottom faces of the concrete, two extensometers with the mirrors at opposite ends being used.

The distribution of strain in the steel joists of specimens Nos. 5 to 12 was obtained in the following manner. Small steel pins were welded or screwed to both flanges as shown in Figs. 1, Plate 1, holes being left in the concrete large enough to provide sufficient clearance round the pins. The

relative movements of the ends of the pins were determined by means of mirror-extensometers. In specimens Nos. 5 to 7 it was possible to measure the strains directly on the unencased flange of the joist. The central deflexions were measured by means of dial-indicators.

Each increment of load was applied for 3 minutes before readings were taken. A prism for determining the stress–strain curve for the concrete

Fig. 2.



METHOD OF TEST.

was made at the same time as the beam and was tested at the same rate. The modulus of elasticity of the steel was taken as $2\,8 \times 10^6$ lb. per square inch, the value obtained from strain measurements on an unencased joist similar to those used in the beams. The actual stress–strain curves for the concrete were used in obtaining the theoretical strains, but the approximate modular ratios at working loads were 7 for mix (*a*) and 6·5 for mix (*b*).

METHOD OF CALCULATING THE STRESSES IN COMPOSITE BEAMS.

The formulas for the stresses in the joist and the concrete are given in Appendix II (p. 111). In obtaining these it has been assumed that there is a planar distribution of strain across the whole section, including the reinforcement; that Hooke's law is obeyed up to the given moment; and that the concrete has a uniform modulus of elasticity, both in tension and compression, over the whole cross section.

The bond stress between the concrete and steel at the upper surface of the joist was estimated by determining the horizontal shearing stress across the section at the level of the top of the flange, treating the beam as though it were homogeneous with the cross section of the steel replaced by an equivalent cross section of concrete.

There may be initial stresses due to shrinkage, and the distribution of stress changes with time owing to shrinkage and creep. These effects do not enter into the above calculations, but will be dealt with in Part II.

The Distribution of Strain across the Section.

The large number of observations taken makes it impracticable to give the individual results in full, and a method of averaging them has been adopted, as shown in Figs. 3, Plate 1. Throughout each of the ranges of load chosen there was no change in the nature of the distribution. The object of taking an average was to decrease the effect of individual errors of observation, although, in fact, these were small, and a diagram of the results at any one load would differ little from the figures given. A result of this method of averaging is that the abscissæ of the points in the figures do not represent the actual strains at a given load. This, however, is immaterial, since only the nature of the distribution is under consideration.

In all the beams, except No. 13 (Figs. 3 (*m*), Plate 1), the distribution of strain in the concrete in compression was approximately planar, but in the tension zone the readings showed an appreciable lag in some of the beams. This was due to cracking, and these readings cannot be relied upon. Readings of the strains in the steel were taken in the tests on beams Nos. 5 to 10. There was a slight difference between the planes of the steel and the concrete in the normal beams Nos. 5, 8, and 9, but the departure from planar distribution across the whole section was too small to be of practical significance.

The split along the web of the joist in beam No. 10 did not materially affect the nature of the distribution of strain across the section (Figs. 3 (*e*)), but in the beam having anchorage angles attached to the top flange of the joist (Figs. 3 (*f*)) the deviation between the planes of the steel and the concrete was in the opposite direction.

The result of destroying the bond between the upper surface of the joist and the concrete is shown in (Figs. 3 (*e*)). The strain in the concrete lagged considerably behind the strain in the steel at high moments, and there was a marked relative movement between the top of the flange of the joist and the concrete. The distribution of strain over the surface of the concrete in the beam reinforced with vertical plates (Figs. 3 (*m*)) was not linear. No explanation of this can be given at present.

The neutral axes were determined from the strains. In the fully encased beams Nos. 1 to 4 and Nos. 8 to 10 they rose with increase of load in the same manner as in the reinforced-concrete beam.[1]

The maximum compressive strains in the concrete were in good agreement with those calculated on the usual assumptions for reinforced concrete, taking the approximate values of the modular ratio deduced from the tests on prisms. The agreement was improved when allowance was made for the creep occurring during the tests by using the curves for the prism-tests [1] (see p. 65).

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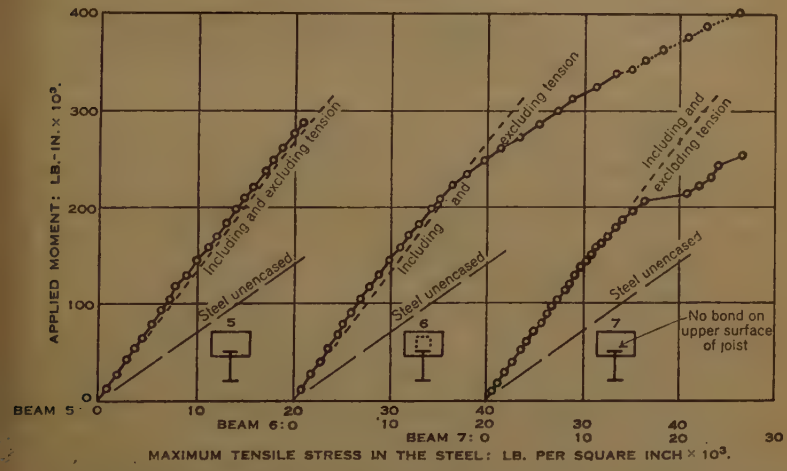
[1] The MS. contains details of the experimental results, and may be consulted in the Institution Library.—Sec. Inst. C.E.

The Stresses in the Steel.

The strains in the steel were determined in beams Nos. 5 to 11, and the maximum tensile stresses in the joists are given in *Figs. 4* and *5*. In the fully encased beams the progressive release of shrinkage stresses as cracking occurred would cause the actual stresses due to the applied load to be less than those shown by the extensometer readings. The maximum error due to this would be about 1,150 lb. per square inch.

The maximum tensile stresses in the joists of the completely encased beams Nos. 8 to 11 were all within the limits of the theoretical "tension" and "no-tension" lines. The allowance for creep makes much less difference to these lines than to those for the compressive strains in the concrete, and they are only slightly curved.

Fig. 4.



The results for the partially encased beam No. 5 are in fairly close agreement with the theoretical line, but not quite so close as the concrete strains. This is in accordance with the slight difference between the planes of the steel and concrete (Figs. 3 (*d*), Plate 1).

In the beam having no bond between the upper surface of the joist and the concrete (No. 7), the results in conjunction with the divergence of the neutral axes (Figs. 3 (*e*)) show the transfer of load to the steel as slip occurs.

The maximum steel strains in the beam having anchorages (No. 6) increased markedly after a moment of about 240,000 lb.-inches had been reached.

Lines have been drawn on the figures to show the maximum tensile stresses in the joist when unencased. It will be noticed that the encasement produced a large decrease of stress in all the joists.

If bond-resistance and anchorage fail in a concrete beam reinforced by

rods, the steel will only be capable of withstanding a small part of the bending moment. The joist in a composite beam under similar circumstances may, on account of its high flexural resistance, take an even greater share of the bending moment than before.

When the steel and concrete act together the joist is subjected to direct

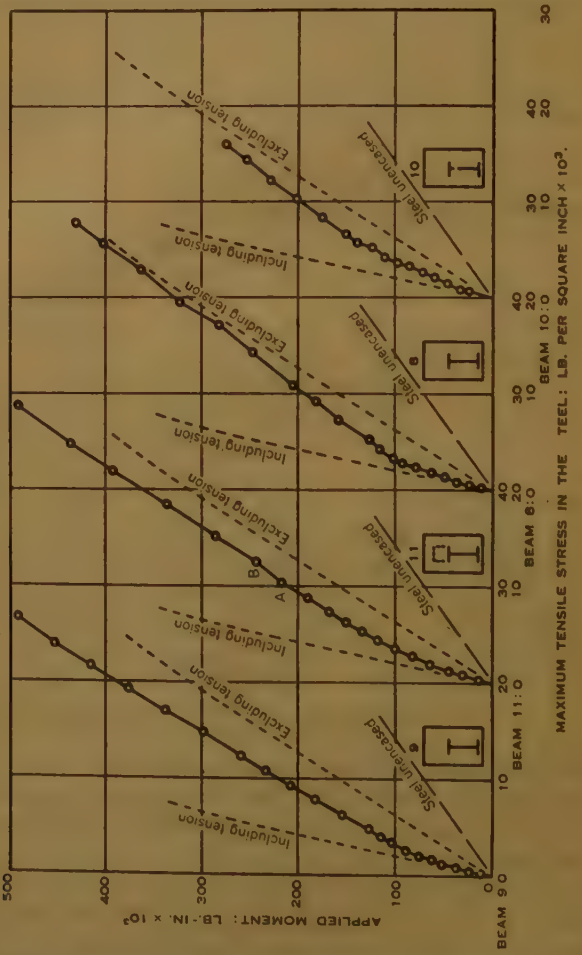


Fig. 5.

tension as well as bending. The theoretical axial stresses (f_d) and maximum flexural stresses (f_b) were calculated by the formulas given in Appendix II, and they are shown as dotted lines in *Fig. 6* (p. 71).

The actual direct and bending stresses in the beam having normal bond but no anchorages were in fairly close correspondence with the theoretical values. They were somewhat below them owing to the slight movement of the steel relative to the concrete.

Beam No. 7, having no bond on the upper surface of the joist, behaved in a similar manner to this beam at low loads, but creep between the steel and the concrete then caused the direct stress to develop more slowly than in beam No. 5. At a moment of 177,500 lb.-inches a slip appears to have occurred followed by renewed resistance, giving an effect similar to anchorage. This was followed by a larger slip and the process was repeated until failure occurred. The maximum flexural stresses rose considerably above those for beam No. 5 owing to the large movement of the steel relative to the concrete (as shown in Figs. 3, Plate 1).

Beam No. 6 behaved in a similar manner to No. 5 until the gradual creep between the steel and the concrete brought the anchorages into action, causing an increase in both the direct and flexural stresses in the steel. The compressive stress in the concrete also increased.

The direct and flexural stresses were determined for the fully encased beams, but are not included in the figures, since, owing to the wide divergence of the two theoretical curves, they cannot be compared directly with the calculated results (*Fig. 5*). The analysis for beam No. 11, however, showed that the kink at AB was almost entirely due to an increase in the direct stress produced by the anchorages, as in beam No. 6. The anchorages in this beam were bolted to the flange of the joist and some creep probably occurred lessening their effect at modern loads. There was also some evidence of actual slip of the anchorages before the beam failed.

The Analysis of the Moments in the Beam.

The bending moment at any section of a composite beam may be regarded as made up of three moments: (*a*) the bending moment in the joist, (*b*) the bending moment in the concrete, and (*c*) the moment due to the axial forces in the steel and the concrete. The theoretical and experimental values of these may be obtained readily if the concrete is uncracked and the distribution of stress in the concrete is planar. These conditions are approximately fulfilled in the partially encased beams until the concrete yields, but the variable and unknown extent of cracking in the fully encased beams makes complete analysis of these impracticable.

The results of the analysis for beams Nos. 5 to 7 are given in Table I (p. 70). The moments obtained from the experimental results for beam No. 5 are in remarkable agreement with the theoretical values when it is taken into consideration that there was a small movement of the steel relative to the concrete (Figs. 3, Plate 1), and, in addition, uncertainties due to the nature of the material and possible inaccuracies in cross section.

There is also a close agreement between the results for beam No. 5 and for beam No. 6 at a moment of 169,000 lb.-inches, before the anchorages had come into action, but at a moment of 260,000 lb.-inches the results for beam No. 6 depart considerably from the theoretical values. It will be

TABLE I.—ANALYSIS OF THE MOMENTS IN THE PARTIALLY ENCASED BEAMS.

Beam.	Applied moment: lb.-inches × 10^3.	Axial force: lb. × 10^3.		Moment due to axial forces: lb.-inches × 10^3.*	Moment in the steel: lb.-inches × 10^3.	Moment in the concrete: lb.-inches × 10^3.	Total moment from experimental results: lb.-inches × 10^3.
		Steel.	Concrete.				
Theoretical	169	21·5	21·5	96·8	44·8	27·4	—
5	169	20·9	20·8	94·8	41·9	25·8	162·5
6	169	20·6	21·4	95·6	42·2	25·1	162·9
7	169	19·4	20·4	90·5	49·1	23·8	163·4
Theoretical	260	33·1	33·1	149·1	69·9	41·0	—
5	260	32·2	32·4	147·1	66·4	39·2	252·7
6	260	37·2	34·4	162·9	75·8	42·0	280·7

* Calculated on the average of the forces in the steel and the concrete.

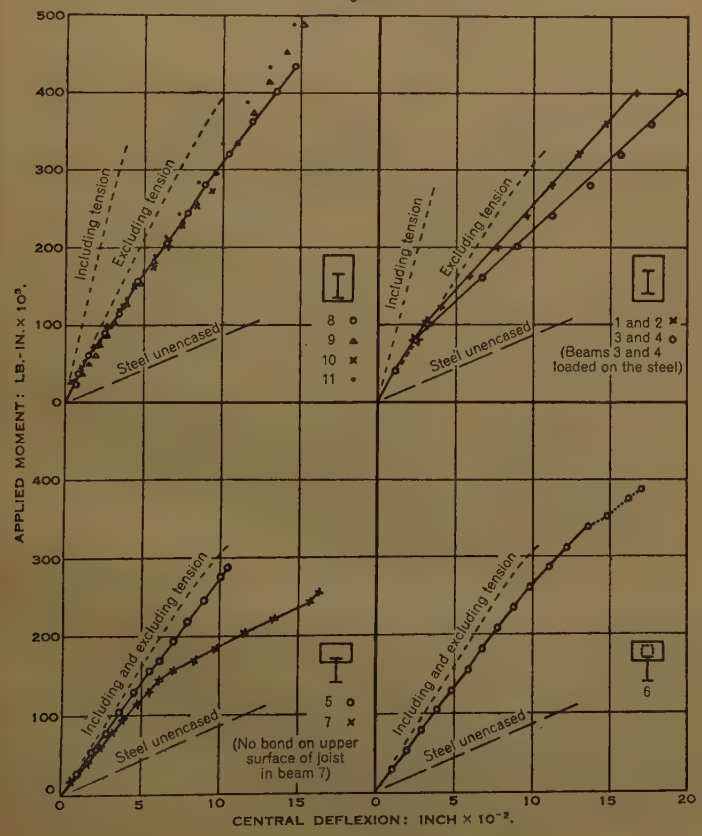
noticed that this is the only case in which the direct forces in the steel and concrete obtained from the experimental results are widely divergent, and in which the sum of the moments is in excess of the applied moment. It is, however, probable that the stresses in the concrete deduced from the experimental results cannot be relied upon owing to the high strains in the concrete. There may also have been hair-cracks in the tension zone. The increase in direct force and bending in the joist due to the anchorages is, nevertheless, clearly indicated.

The effects of slip are shown by the figures for beam No. 7. There was a decrease in the direct force and an increase in the bending moment in the steel, accompanied by a decreased bending moment in the concrete.

Deflexions.

The relations between the central deflexions of the beams (except for Nos. 13 to 15) and the applied moments are shown in *Fig. 6*, together with

Fig. 6.



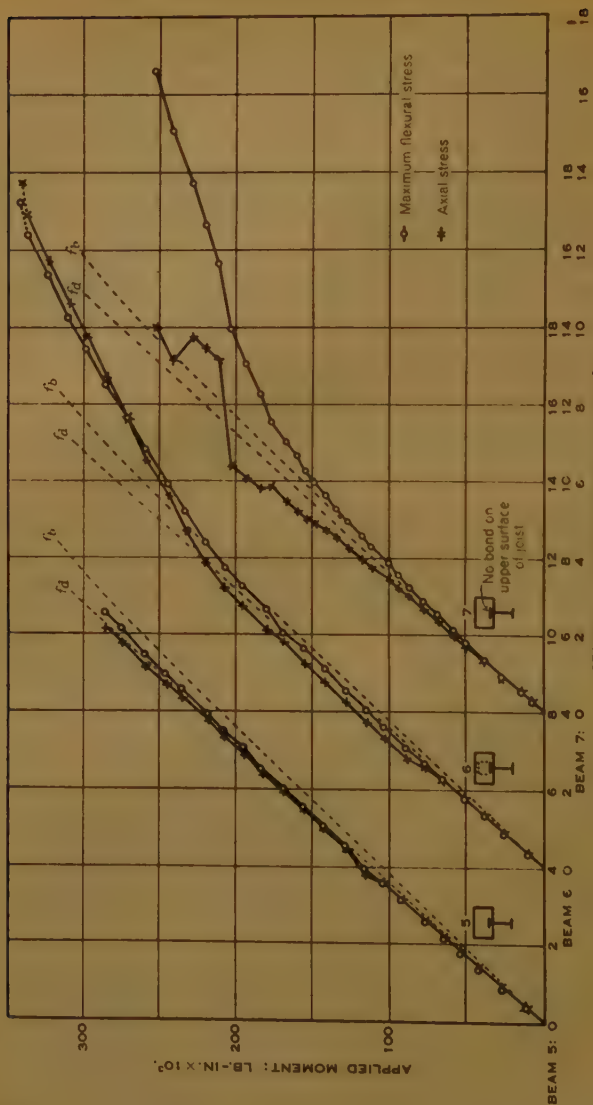


Fig. 7. Axial and Maximum Flexural Stress in the Joist.

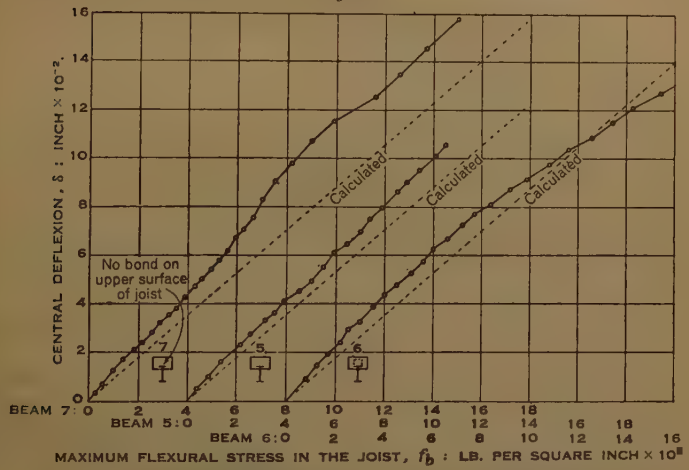
those calculated on the elastic theory using the moduli derived from the tests on prisms. The large reduction in deflexion due to the encasement is clearly shown, but the fully encased beams Nos. 8 to 11 all deflected more in relation to the theoretical curves than the reinforced-concrete beam (No. 15). Above a moment of about 80,000 lb.-inches the deflexions of the beams, Nos. 3 and 4, loaded on the steel were greater than those of the beams loaded on the concrete.

The addition of anchorages in beam No. 6 had little effect upon the deflexion, but the slip in beam No. 7 greatly increased it. The deflexion curve for the latter beam, however, showed none of the irregularities that were observed in the maximum stresses in the steel, since these arose mainly from the large variations in axial stress (*Fig. 7*).

The axial forces in the joist and the concrete do not produce deflexions in these, and therefore the deflexion of the joist due to the bending moment in it is sensibly equal to the total deflexion of the beam. If there is full composite action the bending moment in the joist due to the applied load increases uniformly from the ends to the loading points, and is uniform between the loading points. Thus the central deflexion is, neglecting the deflexion due to shear,

$$\delta = \frac{687 M_s}{E_s I_s},$$

Fig. 8.



RELATION BETWEEN THE CENTRAL DEFLEXION OF THE BEAM AND THE MAXIMUM FLEXURAL STRESS IN THE JOIST.

where M_s denotes the maximum bending moment in the joist and I_s its relevant moment of inertia. Therefore, since the joist is 6 inches deep, the ratio, r, of the central deflexion δ to the maximum flexural stress in the joist, f_b, is

$$r = \frac{\delta}{f_b} = \frac{229}{E_s}.$$

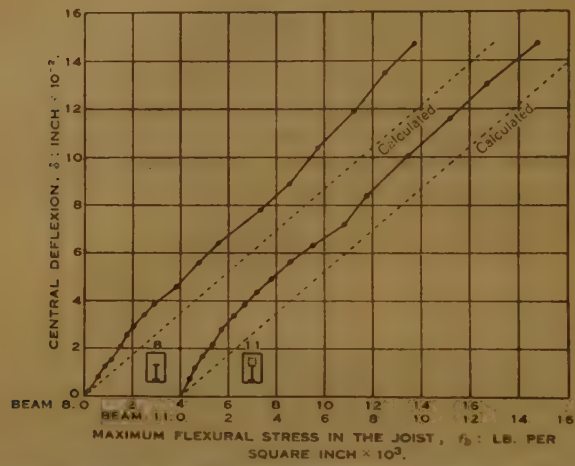
The deflexion due to shear is approximately

$$\frac{13 f_b}{E_s}, \quad \text{giving} \quad r = \frac{242}{E_s}.$$

Straight lines showing this relation are given in *Figs. 8* and *9*, together

with the actual ratios of deflexion to flexural stress determined from the experimental results for beams Nos. 5 to 8 and No. 11. The latter depend only upon the behaviour of the joist, and the comparison between the

Fig. 9.

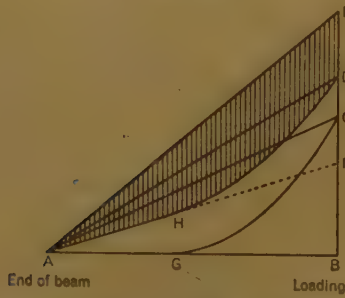


RELATION BETWEEN THE CENTRAL DEFLEXION OF THE BEAM AND THE MAXIMUM FLEXURAL STRESS IN THE JOIST.

theoretical and experimental results is unaffected by any error in the value of E_s, since the same value was used in determining the theoretical curve as in reducing the stresses from the observed strains.

The discrepancies between the calculated and experimental values must

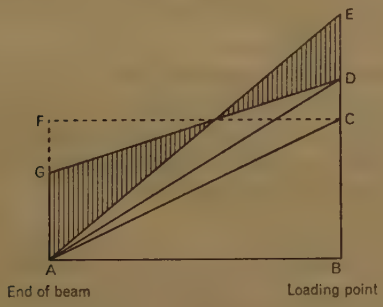
Fig. 10.



therefore be due to variations in the form of the bending-moment diagram for the joist. If BE, *Fig. 10*, represents the total bending moment in the central part of the beam, the theoretical bending-moment diagrams for the outer part, AB, will be the triangles ABC for the moment due to the axial forces in the steel and the concrete, ADC for the bending moment in the

concrete, and ADE for the bending moment in the joist. If, on the other hand, the joist and the concrete acted independently, the bending-moment diagrams for the joist and the concrete would be the triangles AFE and AFB respectively. In each of these cases the form of the bending-moment diagram for the joist is the same, and the relation between the maximum flexural stress in the steel and the central deflexion would be the same. If, however, slip occurred from A to G, allowing complete independence of action, and the bond developed gradually from G to B, giving complete composite action at B, the bending moment due to the axial forces would increase from G to C in the manner shown in the figure, and the bending-moment diagram for the joist would be AHDE. The central deflexion is proportional to the moment about A of the total area of the bending-moment diagram of the joist from A to the centre of the beam. The part of this diagram between B and the centre remains unchanged so long as there is complete composite action in this part of the beam. Since,

Fig. 11.



however, the moment of the area AHDE is greater than that of the triangle ADE, the ratio (r) of the central deflexion to the maximum flexural stress in the joist would be greater under these conditions than it would be with either complete composite action or complete independence of action. The maximum ratio would be reached if there were complete independence of action from A to B, and the axial force suddenly attained the theoretical value for composite action at B. This condition would be approached if there were efficient anchorage at B but no bond from A to B.

If the composite action were not complete in the central part of the beam the points D and C would be lower, but the general argument given above would still hold. Any building up of composite action within this part of the beam would increase the ratio r.

If there were no bond between the joist and the concrete, but they were constrained to bend together by anchorages at the ends, the axial forces would be uniform throughout the beam, and the bending-moment diagram due to them would be AFCB (*Fig. 11*). The bending-moment diagram for the joist under these conditions would be the shaded figure

AGDE. The moment of the area of this figure about A is less than tha of the triangle ADE. Therefore the central deflexion for a given maximun flexural stress would be less than the theoretical value for a composit beam. If there were bond in addition to anchorage the deflexion migh be less, equal to, or greater than the theoretical deflexion depending upo the relative effect of these factors.

The experimental results, *Figs. 8* and *9*, all show the increased valu of the ratio r due to relative movement of the steel and the concrete i the outer parts of the beam. This is particularly marked up to a stress about 10,000 lb. per square inch in the joist (No. 7) having no bond on th upper surface. After this the renewal of axial force referred to in previous section caused a decrease of r. The decrease of r due to en anchorage is shown by the results for beam No. 6 above a stress of abou 8,000 lb. per square inch. The fully encased beam No. 11 had en anchorages only, and these came into action at a lower load than in bean No. 6. The anchorage-angles, however, were bolted to the joist in thi beam, and appear to have crept at higher loads, causing an increase in th value of r above a stress of about 7,000 lb. per square inch.

It will be remembered that the anchorages in beam No. 6 caused considerable increase in the maximum stress in the steel (*Fig. 4*), but tha this was not accompanied by a corresponding increase in the centra deflexion (*Fig. 7*). The reason for this is that the change in distributio of the bending moment along the joist produced by the anchorages caused the value of r to decrease as shown above.

The Development of Surface Cracks.

Typical examples of the manner in which the cracks extended durin the tests are given in Figs. 12, Plate 1. The drawings were made fro photographs of the beams after test. One face only of each specimen i shown, since the disposition of the cracks was usually similar on the tw faces.

The development of cracking followed a normal course in beam No. 15 reinforced by rods and stirrups. The development of the vertical crack was similar in the fully encased joists, but diagonal cracking did not occu until the maximum load was nearly reached.

In the partially encased joists there were only hair cracks between th loading points, but cracks outside the loading points appeared at th maximum load.

The behaviour of the beam reinforced by vertical plates (No. 13) wa interesting. No diagonal cracking took place, showing that the plate formed excellent shear reinforcement, and only in this beam and the bea with ordinary reinforcement were there signs of failure of the concrete i compression.

None of the specimens showed any signs of spalling on the unde

urface of the beam, although there was no mesh or supplementary einforcement.

The Failure of the Beams.

The moments at failure, including the moment due to the weight of he beam, are given in Table II (p. 78), together with the nominal bond stress alculated from the formula given in Appendix II, neglecting the concrete n tension. The modular ratios were taken as 7 for mix (*a*) and 6·5 for nix (*b*), which are approximately in accordance with the prism-tests. The rdinary method for reinforced concrete was used for calculating the bond tress in beams Nos. 14 and 15.

The reinforced-concrete beam No. 15 failed by yield of the steel in ension, followed by crushing of the concrete on the compression face. he specimen with the vertical plates (No. 13) also failed owing to excessive eformation of the lower edges of these plates followed by crushing of he concrete, but not until a considerably higher moment was reached, lthough the calculated working moment was somewhat lower than in eam No. 15. This was no doubt due to the gradual yield of the plates. he yield of a steel joist is also gradual, and consequently, in a composite eam, if failure in bond and diagonal tension do not occur, there is a reater reserve of strength than in a reinforced-concrete beam having the ame maximum tension in the steel.

In the fully encased composite beams Nos. 1 to 4 and Nos. 8 to 9, in hich no special means were adopted to anchor the steel to the concrete, ailure was accompanied by the opening of diagonal cracks in the concrete utside the loading points. This indicates that the failure was caused ther by diagonal tension or by diagonal tension following a breakdown f bond.

In beams Nos. 8 and 9 the relative movement of the joist and the oncrete was observed by means of dials. No appreciable movement took lace until the maximum load was nearly reached. The diagonal cracks id not open until after the relative movement was observed, and this dicates that the primary cause of failure was breakdown of bond.

This was confirmed by the test on specimen No. 11, which had small nchorage-angles bolted to the top flange of the joist near to the ends of he beams, since failure did not occur until a much higher load was reached han in beams Nos. 8 and 9. The dials indicated a relative movement etween the steel and the concrete at the failing load due to slip of the olts.

The increase of strength produced by anchorages is also illustrated y the results obtained in the tests on the partially encased beams Nos. 5 nd 6. Failure in beam No. 5 was undoubtedly due to slip. The angles elded to the top flange of beam No. 6 prevented this, and it supported considerably greater load than the other partially encased beams. The

TABLE II.—RESULTS OF THE TESTS TO FAILURE.

Beam No.	Concrete.	Working moment (Code of Practice): lb.-inches × 10^3.	Total moment at failure: lb.-inches × 10^3.	Load-factor on failure.	Nominal bond stress at failure: lb. per square inch.	Cause of failure.
1	Mix (*a*)	140·5	527	3·75	322	Slip followed by diagonal tension.
2	,,	147·5	579	3·92	346	
3*	,,	145	475	3·28	289	
4*	,,	143	475	3·32	289	
5	,,	136·5	359	2·63	224	Slip.
6†	,,	136·5	510	3·74	—	Excessive deformation of steel.**
7‡	,,	136·5	313	2·29	195	Slip.
8	Mix (*b*)	198	488	2·47	282	Slip followed by diagonal tension.
9	,,	198	501	2·53	289	
10	,,	198	319	1·61	183	Slip.
11§	,,	198	708	3·58	—	Slip of the anchorages followed by diagonal tension.
12	,,	285·5	540	1·89	233	Diagonal tension and slip.
13	,,	181	866	4·79	—	Excessive deformation of steel and failure of concrete in compression.
14	,,	193·5	332	1·72	137	Slip of horizontal plate.
15	,,	187	708	3·78	—	Yield of horizontal bars and failure of concrete in compression.

* Beams Nos. 3 and 4 loaded and supported on the steel.

† Beam No. 6 similar to beam No. 5, but with welded anchorage-angles.

‡ Beam No. 7 similar to beam No. 5, but having no bond on upper surface of joist.

§ Beam No. 11 similar to beam No. 8, but with bolted anchorage-angles.

** This test was stopped when the central deflexion was about ½ inch. Subsequent tests indicate that the ultimate load had not been reached.

The working moment on an unencased joist, laterally secured, of the same section as those used in the tests is 126,000 lb.-inches for a working stress of 18,000 lb. per square inch.

est was not carried to complete failure, but a load about 50 per cent. in excess of that producing yield of the steel was reached.

In all these beams the neutral axis was close to the level of the top of the joist. Compression rods were added in specimen No. 12 in order to bring the neutral axis above the joist, thus increasing the liability to failure by diagonal tension of the concrete and decreasing the bond stress on the top of the joist. When tested, the diagonal cracks opened before slip was registered by the dials. Thus the primary cause of failure was diagonal tension.

Failure due to breakdown of bond also occurred in the specimen having a horizontal flat plate as reinforcement (No. 14). The plate was not anchored in any way, and this would account to some extent for the lower value of the bond at failure as compared with that in the specimen reinforced with bars, but, even taking this into consideration, 137 lb. per square inch is a low figure. Examination of a number of broken specimens showed that, whilst the surface of the concrete which has been in contact with the upper surface of a joist, or with a vertical surface such as the web of a joist, is smooth, the accumulation of water under the lower surface of a joist or flat plate during pouring causes the surface of the concrete to become overlaid with networks of minute ridges, probably of neat cement, which form the only contact between the steel and the concrete and offer little resistance to relative movement of these. It follows from this, and from other evidence obtained in the experiments, that the bond at the upper surface of a joist or plate and also at a vertical face is good, but that it is bad at all under-surfaces.

The importance of continuity of section of the joist is illustrated by the results obtained in the test on the beam with a split joist (No. 10). Slip occurred at a much lower load than in the similar beams, Nos. 8 and 9, in which the joist was intact, although, as shown in the previous sections, this beam had behaved in a similar manner to Nos. 8 and 9 until the failing load was nearly reached.

Higher bond stresses were reached in the fully encased beams Nos. 1 and 2 than in the partially encased beam, No. 5, showing that the concrete on the tension side of the neutral axis contributes to the bond resistance.

Considering the conventional nature of the calculations, the different mixes used, and the different position of the joist in the cross section of the beam, the bond stresses for the fully encased beams Nos. 1, 2, 8, and 9 are reasonably consistent. They would be in better agreement if, as in practice, a higher value of the modular ratio were used. For example, a value of 15 would give results ranging from 268 to 308 lb. per square inch.

The bond stress at failure will depend to some extent upon the position of the joist in the concrete, but, from the evidence obtained in these tests, it seems that a reasonable lower limit is about 240 lb. per square inch for a fully encased beam. With a factor of safety of 4 this gives 60 lb. per

square inch as the working stress in bond. For a partially encased beam 50 lb. per square inch would give about the same factor of safety.

These figures are low compared with those in common use for reinforced concrete, but in many cases the bond need not be considered. Also, if bond does govern, it has been shown that the addition of anchorages welded to the joist prevents failure due to breakdown of bond. This procedure has been recommended by L. Baes [1], R. A. Caughey [2], and others. In many cases in practice the beams will be attached to other members at the ends, and further anchorage may be unnecessary. This is confirmed by the tests which will be described in Part II.

The results on the beam with compression rods show that, in certain cases, diagonal tension may govern, and additional shear reinforcement would then be necessary.

If failure due to breakdown of bond or diagonal tension in the concrete is prevented, the maximum load will be reached when the steel deformation becomes excessive and the concrete breaks down in compression. This only occurred in the reinforced-concrete beam (No. 15) and in the beams with vertical plates (No. 13), but if the loading of beam No. 6 had been continued it would either have failed in this manner or by the tearing of the concrete at the anchorages.

In each of these beams, the steel yielded before the ultimate load was reached, and the large reserve of strength after yield in beams Nos. 6 and 13 has already been noted.

The working moments calculated in accordance with the Recommendations for a Code of Practice for Reinforced Concrete [3], and the corresponding load-factors on failure, are given in Table II. These factors compare satisfactorily with that of the reinforced-concrete beam for beams Nos. 1 and 2, owing to the low bond stress at working load in these beams. In beams Nos. 8 and 9 the bond stress was higher, and the load-factors are low, but the results for the beams having anchorages show that these restore the margin of safety. It should be remarked, however, that the anchorages in beam No. 6, by increasing the tension in the steel at high loads, caused yield of the steel to occur at a lower load than it would have done in the beam without anchorages (No. 5) if failure by slip had not taken place in that beam before the steel yielded. From this it would appear that anchorages may sometimes decrease the load-factor on yield. This point requires further investigation.

Summary and Conclusions. Part I.

(1) The tests confirmed that, if the bond is sufficient to ensure composite action, the stresses in composite beams without anchorages or end con-

[1] *L'Ossature Métallique.* January–February, 1933, p. 13.

[2] R. A. Caughey. "Reinforced Concrete." London, 1937, p. 159.

[3] Report of the Reinforced Concrete Structures Committee. H.M. Stationery Office, 1933.

nexions may be estimated by the methods used for reinforced concrete, although small relative movements of the planes of the steel and the concrete were observed. The deflexions of all the beams were greater than those calculated on the usual assumptions for reinforced concrete. Cracks developed in the fully encased beams in a similar manner to those in the reinforced-concrete beam, except that diagonal cracking was delayed until failure was almost reached.

(2) Anchorage-angles welded to the top flange of a partially encased joist did not affect the deflexion, but a test on a beam having no bond on the upper surface of the joist showed that slip greatly increased it.

(3) The joist in a composite beam is, in general, subjected to axial and flexural stresses. The flexural resistance of the joist is much greater than that of ordinary reinforcement, and may be a considerable proportion of the total resistance to bending.

The axial and flexural stresses were obtained from the experimental results, and compared with the theoretical values. The experimental and theoretical stresses were in close agreement for the partially encased joist without anchorages. Anchorage-angles on the top flange of the joist were found to cause an increase in both axial and flexural stresses, whilst slip caused an increase in the flexural stress and a decrease in the axial stress.

(4) The concrete encasement is also under axial and flexural stresses, the axial force being equal and opposite to the axial force in the steel. When the concrete was uncracked the experimental results enabled the actual partition of the moments to be determined. This was in close agreement with the theoretical partition for the partially encased joist without anchorages. Slip was found to cause a decrease in the bending moment in the concrete and the moment due to the direct forces, accompanied by a corresponding increase in the bending moment in the steel.

(5) The deflexion of a composite beam is the same as that of the encased joist, and must be sensibly that due to the bending moment in the joist. The ratio between the central deflexion and the maximum flexural stress in the joist was determined theoretically on the assumption that plane sections remained plane. It is shown that, if the axial force is tensile, slip increases this ratio and end anchorages or connexions decrease it. This was confirmed from the experiments. Application of these results enables the interaction between the joist and the concrete to be followed.

(6) The above analysis showed that in all the beams, there was some relative movement between the steel and the concrete before either bond or anchorage came into play.

(7) The differences observed between the theoretical and actual deflexion-curves of a composite beam are due to the changes in the shape of the bending-moment diagram of the joist, arising from the effects considered in the last two paragraphs. There is a corresponding change in the shape of the bending-moment diagram for the concrete, and similar changes must occur in ordinary reinforced-concrete beams.

(8) The failure of the normal composite beams, in which no special means were adopted to anchor the steel to the concrete, was due to break-down in diagonal tension following slip between the joist and the concrete. In one specimen compression rods were added in order to decrease the liability to failure by slip. This beam failed by diagonal tension.

The addition of anchorage-angles considerably increased the ultimate strength.

A high load was also reached before failure occurred in the beam having vertical plates as reinforcement, since these prevented failure by diagonal tension.

(9) The experiments, in conjunction with those described in Part II, show the importance of efficient anchorages or end connexions, and it is suggested that, if these cannot be relied upon, the bond stress should not exceed 60 lb. per square inch for a fully encased joist and 50 lb. per square inch for a partially encased joist.

(10) The bond between a vertical plate and concrete is usually good, but it is weak on the underside of a horizontal plate.

PART II.—EXPERIMENTS UNDER SUSTAINED LOADING.

Description of the Specimens and Method of Testing.

The beams were of two types, as shown in *Fig. 13*. It was necessary to use joists of small section, 4 inches by $1\frac{3}{4}$ inch, in order to make loading by dead weights practicable. Some difficulty was experienced in obtaining these small joists true to form, but the ones finally used in the experiments were satisfactory. T-beams were adopted in order to obtain a better balance between the steel and concrete stresses than in the beams employed for the direct-loading tests.

The end connexions shown in the figure were introduced in order to simulate the conditions of anchorage that would be present in a structure without introducing appreciable end restraints.

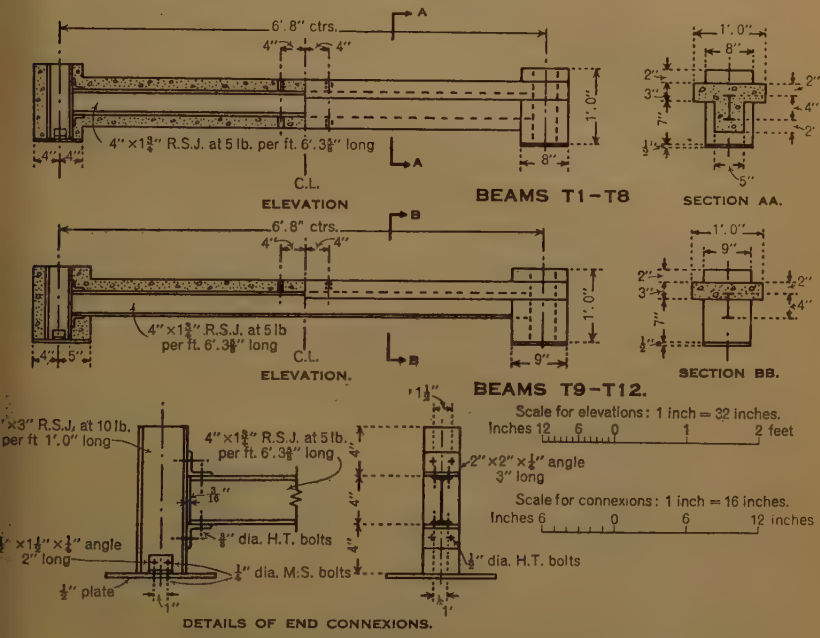
The shuttering in beams Nos. T1 to T8 was suspended from the joist by hangers during the pouring of the concrete, the beams being supported at the ends. The maximum tensile stress produced in the steel by the weight of the shuttering and wet concrete was 3,300 lb. per square inch. In the remaining beams the shuttering was supported during and after pouring.

The same mix of concrete was used in beams Nos. T1 to T8 as in beams Nos. 1 to 7 (Part I). For the remaining beams the water/cement ratio by volume was decreased to 0·66, and this mix will be referred to as mix (*c*). The compressive strength of this concrete was 6,300 lb. per square inch at 28 days and the prism-modulus 5×10^6 lb. per square inch.

The beams were kept moist for about 12 days, and the tests were commenced about 28 days after pouring.

The span of the beams was 6 feet 8 inches and equal loads were applied at 2 feet 2 inches from each support. One end of the beam rested on a half-round bar and the other on a roller. The loads were applied in the following manner. Vertical rods were attached to the ends of a roller which rested on the beam at the loading point, a bearing plate being inserted between the roller and the beam. The platform for carrying the loads was

Fig. 13.



DETAILS OF THE BEAMS (PART II).

supported by large nuts threaded on to the lower ends of the rods. In order to avoid shock, the platform was packed underneath when the loading material was being placed in position. It was then raised by rotating the supporting nuts on the vertical rods, until it was clear of the packing, thus gradually transferring the load to the beam.

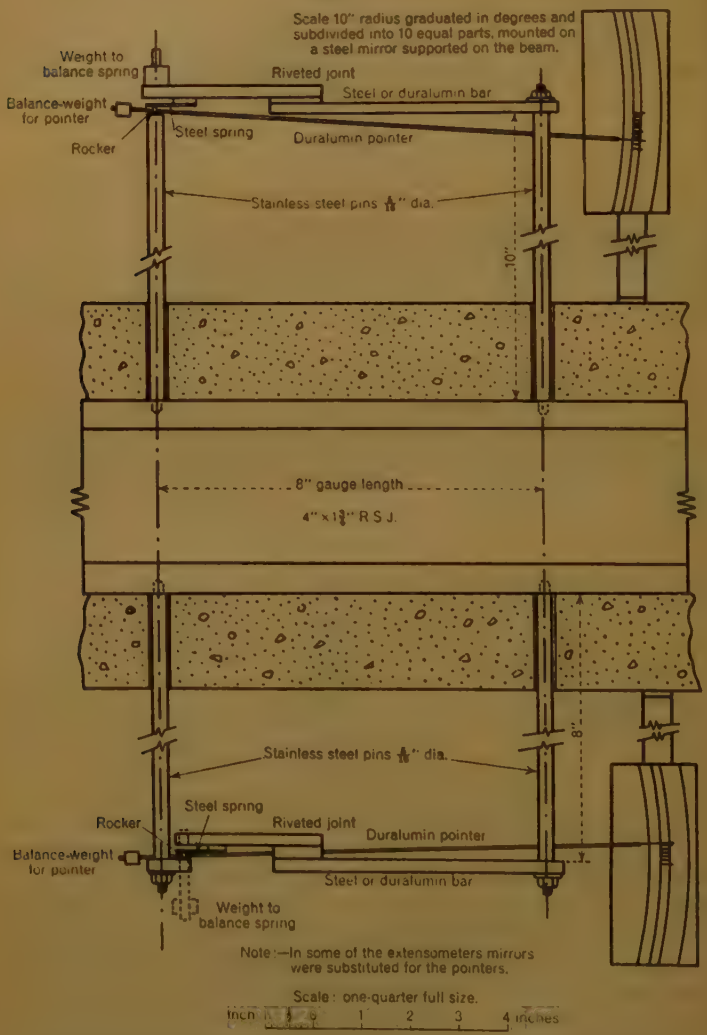
The central deflexions of the beams were measured by micrometers reading to 0·0001 inch. A small stainless steel plug was inserted in the top of the beam under the micrometer in order to make contact more definite. An optical recording deflectometer was employed for the beam tested in a constant-temperature room.

Beams Nos. T1 to T3 and T6 to T8, tested in the laboratory, were

supported on rigid steel frameworks, and beams Nos. T4 and T5, tested out t
of doors, were carried on a steel framework resting on brick pillars.

The strains in the steel were measured by means of extensometers of t

Fig. 14



EXTENSOMETERS USED IN THE TESTS UNDER SUSTAINED LOADING.

the type shown in *Fig. 14*. The pointers were only used in the open-air tests. In these tests the extensometers and micrometers were protected by wooden boxes, the lower one having a glass front through which readings

could be taken and the upper one a door in order that the micrometer could be read.

Thermometers touching the steel were embedded in the end-pieces of the beam to indicate the temperature of the joist, and thermometers in the boxes recorded the temperature of the extensometers. In the tests in which duralumin extensometer-bars were used it was necessary to correct for the difference between the expansion of the steel and the duralumin, when readings were taken at a temperature different from that of the initial readings. When the temperature of the joist was different from that of the extensometers it was also necessary, with either steel or duralumin bars, to correct for this. The latter correction was only made for the beams tested out of doors.

The coefficient of expansion of steel was taken as $0{\cdot}61 \times 10^{-5}$, and the difference in expansion between the steel and duralumin was determined by experiments on an unencased joist. It was found that an error of 1° F. in the difference of temperature used for either correction would lead to an error of about 200 lb. per square inch in the recorded stress.

The partially encased beams were supported side by side on steel frames carried on concrete blocks. The span of the beams was 6 feet 8 inches, and the loads were applied at 2 feet $2\frac{1}{2}$ inches from the supports. The loads were made up of concrete beams remaining from previous tests, and steel billets. The beams were lowered gradually, by means of travelling cranes, on to rollers resting on the beam under test at the loading points, bearing plates being inserted between the beam and the rollers.

The strains at the top of the joists were measured by means of extensometers of similar pattern to those already described, with mirrors in place of pointers. The strains at the bottom were determined by mirror-extensometers of the usual type, and these were also used, arranged vertically between the joists and the supporting frames, to obtain the central deflexions of the joists. A reference-mirror attached rigidly to the support was placed by the side of each of the movable mirrors. A telescope and scale were mounted on a horizontal steel rod, perpendicular to the beams. Initial scale-readings were taken in all the fixed and rotating mirrors before the beams were loaded, the telescope and scale, which were fixed relatively to one another, being moved along the rod into a suitable position for each rotating mirror with its corresponding fixed mirror. For each subsequent reading of the rotating mirrors, the telescope and scale were reset so as to give the initial reading in the corresponding fixed mirror.

Temperature-corrections, as described above, were made for the extensometers on the upper flanges of the joists, and the deflexion-readings were corrected for the expansion and contraction of the vertical extensometer bars. It was unnecessary to correct for any vertical movement of the fixed mirrors, since this would not affect the readings.

Particulars of the tests are given in Table III. In addition to the

TABLE III.—PARTICULARS OF THE FIRST SERIES OF TESTS CARRIED OUT UNDER SUSTAINED LOADS.

Beam No.	Type.	Concrete.	Applied moment: lb.-inches $\times 10^3$.	Conditions of storage.	Remarks.
T1	Fully encased.	Mix (*a*)	0	Laboratory.	—
T2	,, ,,	,,	25·2	,,	—
T3	,, ,,	,,	50·2	Constant-temperature room.	Approximate working load.
T4	,, ,,	,,	50·2	Outdoors.	Approximate working load.
T5	,, ,,	,,	50·2	,,	Approximate working load.
T6	,, ,,	,,	50·2	Laboratory.	Alternately dry for 7 days and wet for 7 days.
T7	,, ,,	,,	50·2	,,	Alternately dry for 14 days and wet for 14 days.
T8	,, ,,	,,	50·2	,,	Alternately dry for 28 days and wet for 28 days.
T9	Partially encased.	Mix (*c*)	0	Laboratory.	—
T10	,, ,,	,,	21·2	,,	—
T11	,, ,,	,,	42·4	,,	—
T12	,, ,,	,,	63·6	,,	—
T13	Partially encased. No end anchorage.	,,	42·4	,,	—

tests given in the Table direct-loading tests to destruction were carried out on similar beams for purposes of comparison.

The readings of strain and deflexion were taken daily for a period of from 100 to 150 days, and after this at intervals of a few days. The observations on the beams tested out of doors were made in the morning before the rays of the sun could reach the beam. The daily rainfall given in *Figs. 16* and *17* (p. 88) was obtained from the official figures of the Edgbaston Observatory. The humidity of the atmosphere in the constant-temperature room and the concrete laboratory was observed by means of a recording hygrometer, and the temperature by means of a recording thermometer.

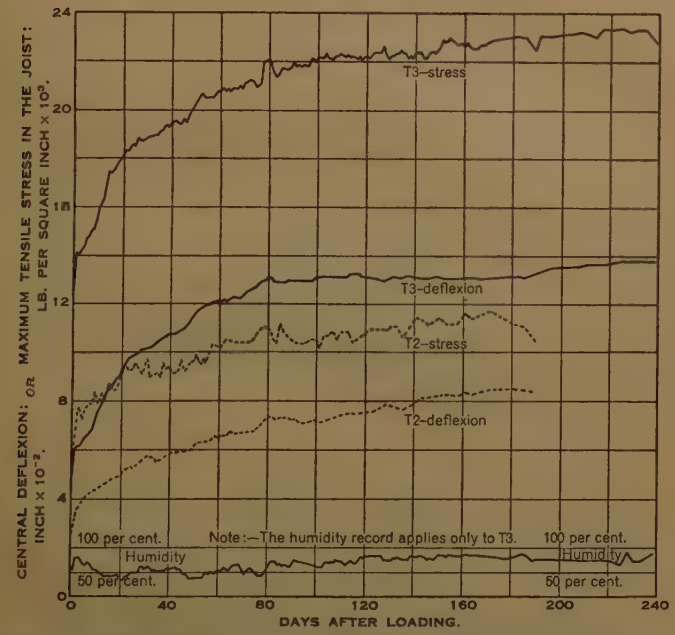
The temperature of the room in which beam No. T3 was tested was kept, as far as possible, at 62° F. $\pm$ 1° F. by means of a thermostat. Occasionally the control failed and the temperature decreased for a short time. On the other hand, it sometimes increased in hot weather, the maximum temperature recorded being 68° F. For the greater part of the period of the test, however, it remained constant.

The Increase of Deflexion and Maximum Tensile Stress in the Steel with Time.

The observed deflexions of beams Nos. T2 to T5 and the tensile stresses at the lower surface of the steel are shown in *Figs. 15*, *16* and *17*. The record of humidity is given for the beam tested in the constant-temperature room and the rainfall and temperature for the beams tested out of doors (Nos. T4 and T5).

Many observers have recorded the increase in deflexion of reinforced-

Fig 15.



Central Deflexions and Maximum Tensile Stresses in the Steel for Beams Nos. T3 (Constant-Temperature Room) and T2.

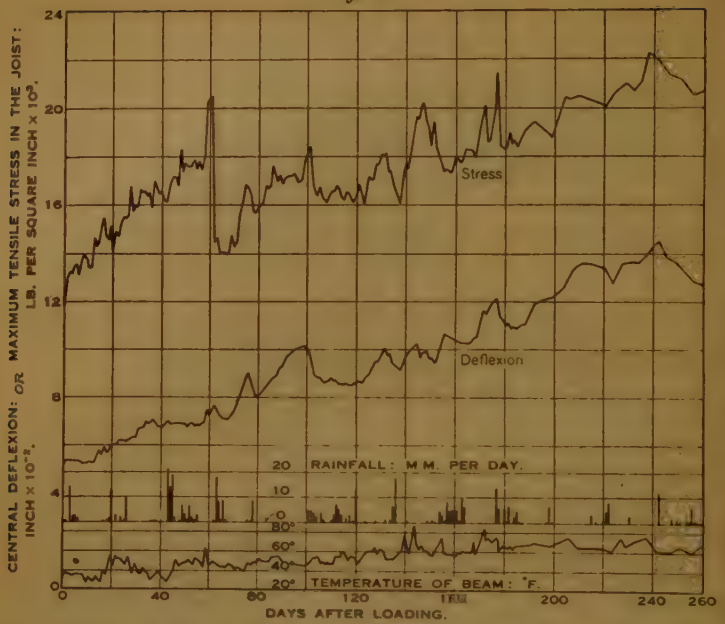
concrete beams with time, the rate of increase becoming less marked as the beam ages. A few examples may be given for comparison with the results obtained in the experiments on encased joists.

W. K. Hatt [1] and Dr. Oscar Faber [2], using simply-reinforced beams of quite different proportions, both found the ratio of the deflexion of a beam 150 days after loading to its deflexion on loading to be between

[1] W. K. Hatt, "Effect of the Time Element in Loading Concrete." Proc. Am. Soc. Test. Mat., 1907, p. 421.

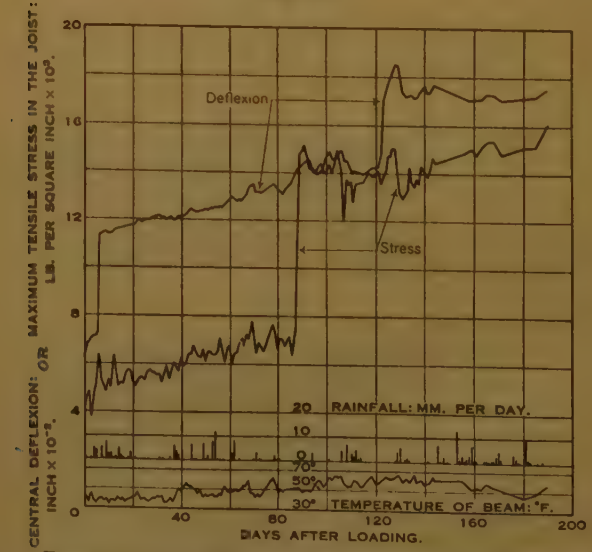
[2] Oscar Faber, "Plastic Yield, Shrinkage, and other Problems of Concrete, and their Effect on Design." Minutes of Proceedings Inst. C.E., vol. 225, p. 27.

Fig. 16.



CENTRAL DEFLEXION AND MAXIMUM TENSILE STRESS IN THE STEEL FOR BEAM NO. T4 (OUT OF DOORS).

Fig. 17.



CENTRAL DEFLEXION AND MAXIMUM TENSILE STRESS IN THE STEEL FOR BEAM NO. T5 (OUT OF DOORS).

·5 and 3·0. In 1932 M. H. Bakir, a research student in the Civil Engineering Department of the University of Birmingham, using 7-inch by 4-inch beams reinforced by two rods $\frac{5}{16}$ inch diameter, hooked at the ends and loaded at the third points, obtained values ranging from 2·72 for lightly loaded beams to 2·4 for beams carrying loads somewhat greater than the working loads.

The ratios at 150 days for the fully encased joists in the present tests were 2·65 for No. T3 and 3·05 for No. T2, which only carried half the load. All these values, on very different specimens, only range from 2·40 to 3·05. This approximate agreement may be merely fortuitous, but it is remarkable.

The behaviour of the beams tested out of doors was much more irregular than that of beams Nos. T2 and T3 owing to the effect of the weather. The ratios reached after 150 days were 2 and 2·8 for beams Nos. T4 and T5 respectively.

Wet periods retarded the growth of the deflexion, and, after 150 days, the values were only 1·51, 1·6, and 1·68 times the original deflexion for beams Nos. T6, T7, and T8 respectively.

In beams Nos. T10 and T11 only the upper flange was encased, and the increase of deflexion was somewhat less than in No. T2 and No. T3, the ratio being 1·9 after 150 days for each beam. This is reasonable, since part of the increase of deflexion in the fully encased beams was undoubtedly due to the development of tension-cracks.

It should be noted that hair-cracks developed in beams Nos. T3 and T4 on loading, but they were not observed in beam No. T5 until 20 days after loading. The cracks developed, and further tension-cracks appeared as the test continued, but the width of the cracks remained small throughout the tests.

There appears to be little published evidence concerning the increase in the tensile stress in the rods of a reinforced-concrete beam under sustained loads, but Dr. W. H. Glanville [1] gave the results of a test on a simply-reinforced beam loaded so that the calculated stress in the steel was 6,000 lb. per square inch and the maximum compressive stress in the concrete was 1,500 lb. per square inch. In this beam the steel stress increased with time in a similar manner to the maximum tensile stress in beam No. T3 (*Fig. 15*), reaching about 1·4 times its value on loading after 50 days. The maximum tensile stress in beam No. T3 after the same period was 1·88 times its value on loading. The corresponding figures for the partially encased beams were 1·40, 1·42, and 1·32 in beams Nos. T10, T11, and T12 respectively.

The curves of change of stress with time are approximately of the same form as those of change of deflexion in the beams tested in the laboratory. This also applies to beam No. T4, tested out of doors, although the change of stress was much more irregular, but in beam No. T5 a remarkable

[1] W. H. Glanville and F. G. Thomas, "Creep of Concrete under Load." Journal Inst. Struct.E., vol. 11 (1933), p. 62.

sudden increase of stress took place after 89 days. This beam did no crack on loading, and the stress reached after the sudden increase occurre was still somewhat less than in beam No. T4 at the same age. Thi irregularities of behaviour will be discussed further in a later section.

Although the readings on beam No. T5 were discontinued after 190 day the beam remained loaded, and the maximum stress was read after thi load had been applied for about 2½ years. It had then reached 23,200 l per square inch, which is 1·6 times the calculated value, omitting thi concrete in tension, at the time of loading.

Effect of Changes in Humidity and Temperature.

It is well known that changes of humidity affect the shrinkage o concrete, and the maximum effect occurs when the concrete is alternatel dried and wetted. These effects influence the deflexion of the beam an also the stresses in the reinforcement in a marked manner. M. Suquet measured the shrinkage-stresses in steel tubes embedded in unloade rectangular prisms. He found that the shrinkage-stress decreased rapidl when the prism was wetted, and increased again slowly when the prism was left to dry.

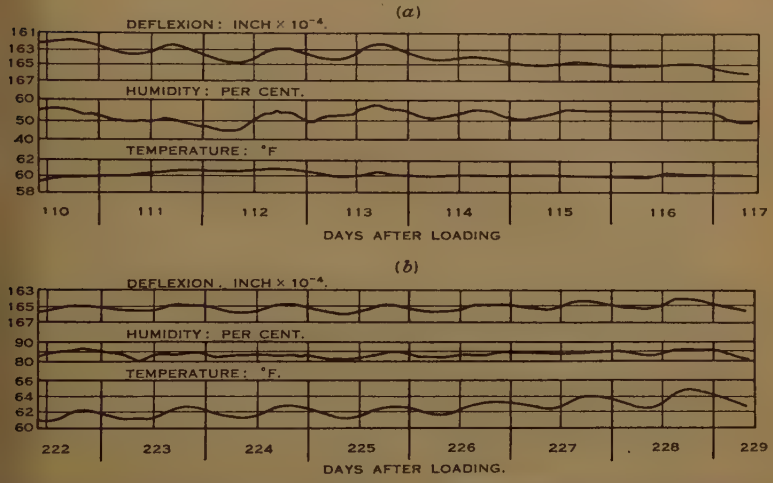
M. H. Bakir, in the course of the experiments referred to previously obtained a continuous record of the deflexion of a reinforced-concrete beam under load, kept in the constant temperature room, together with the humidity. The results showed that if the humidity had remained constant the deflexion-record would have been a smooth falling curve. The actua record, however, showed fluctuations which bore a close relation to the changes of humidity. Examples of the curves are given in *Fig. 18(a)*. When the humidity fell the deflexion increased, and *vice versa*. There was also an effect upon the mean curve. This was shown in an exaggerated form by an experiment in which a loaded beam was, after a period o exposure to the atmosphere, totally immersed in water. The immersion reduced the general rate of increase of deflexion to about one-third of that of a similar and contemporary beam kept exposed to the air in the same room.

Several times during the course of the experiment in the constant temperature room the temperature-control and insulation of the room was insufficient to prevent fluctuations of temperature, and sometimes this fortunately occurred during a period of constant humidity. It was then found that the fluctuations in the deflexion-curve followed with remarkable closeness the changes of temperature, the deflexion increasing with decrease of temperature, and *vice versa* (*Fig. 18(b)*).

[1] M. Suquet, "Note sur la détermination des efforts occasionnés dans les armature d'une poutre en béton armé par le retrait du béton." *Annales des Ponts et Chaussées* 1932–III, p. 337.

These results were borne out by the tests on the encased beams, and he observations on beam No. T3 also showed the effect of the general evel of humidity. The deflexion and humidity for this beam are shown n *Fig. 15*. The scale is too small to show clearly the fluctuations of eflexion with humidity, but the effect of the percentage humidity upon he general rate of increase can be seen. During periods of high humidity he rate of increase of deflexion was much less than during periods of low umidity.

Fig. 18.



Effect of Variations in Humidity and Temperature on the Deflexion of a Reinforced-Concrete Beam.

The record of the stresses given in the same figure shows that the umidity also has an effect upon the rate of increase of stress in the lower ange of the joist, roughly in correspondence with the effect upon the eflexion. This was also observed in the partially encased joists.

Beams in the open air are not only subjected to the ordinary changes f temperature and humidity, but also to wetting from rain and uneven eating due to the direct rays of the sun. These cause irregularities of ehaviour (*Figs. 16* and *17*) which will be discussed later.

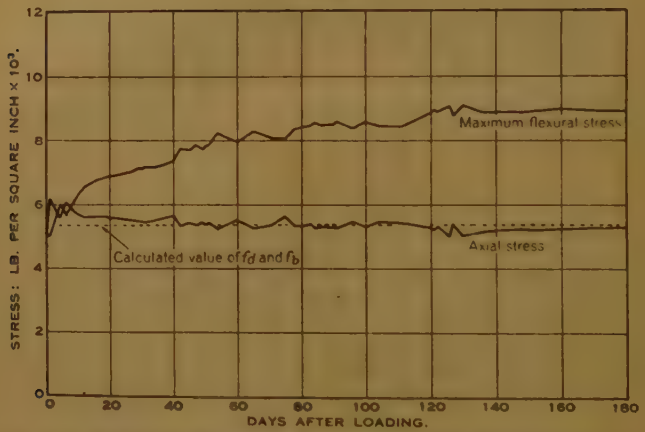
Analysis of the Stresses and Deflexions.

The methods of analysis of the stresses and deflexions given in Part I ay usefully be applied to the tests under sustained loading.

It will be most convenient to commence with the partially encased joist o. T11, since no cracking occurred. The axial and maximum flexural resses are shown in *Fig. 19*. In this beam the theoretical axial and

flexural stresses are equal and the actual values were close to the theoretic value at the beginning of the test. The axial force increased during t first day, but after a few days, except for minor irregularities, it remain close to the theoretical value throughout the test, the moment due to t axial forces only rising from 43·1 per cent. to 45 per cent. of the tot moment in the first 150 days. The main effect of the sustained loadi was, therefore, a gradual transfer of the bending moment in the concre to the joist. The explanation of this is that, if there is composite actio the value of the moment due to the axial forces varies little with t modular ratio.

Fig. 19.



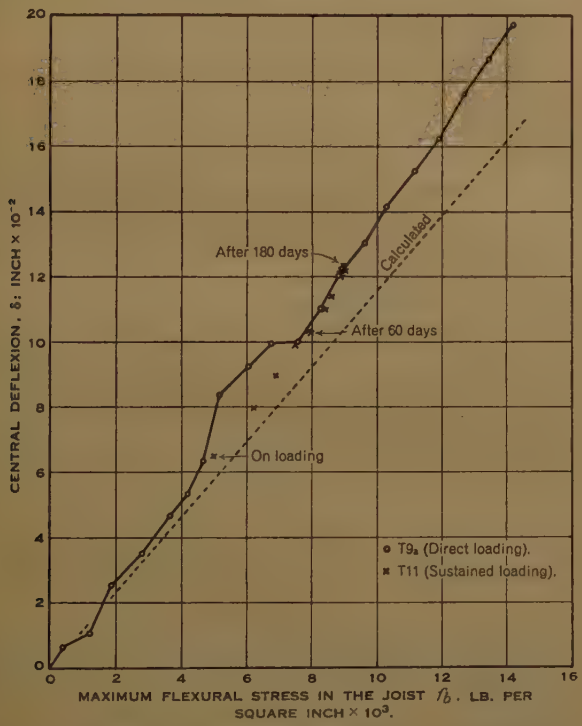
Axial and Flexural Stresses in the Joist for Beam No. T11.

A direct-loading test was carried out on a beam, No. T9*a*, similar t No. T11, and the relation between the central deflexion and the maximu flexural stress is given in *Fig. 20*. It was shown in Part I (pp. 74 *et seq* that, when the experimental points are above the theoretical line, slip of th joist has occurred, and that when they are below it some of the direct forc is transferred from the steel to the concrete by the end connexions. Th curve for beam No. T9*a* is similar in form to that obtained for the partiall encased beam without anchorages (Part I, *Fig. 8*, p. 73) except that sli appears to have occurred at a stress of about 4,100 lb. per square inc followed by action of the end connexions and renewed grip. Points on th corresponding curve for beam No. T11 are also shown on the figure. Apa from the increased deflexion due to slip in beam No. T9*a* the two curv are in remarkable agreement. This indicates that the increased flexur stresses caused by creep of the concrete were, except when slip occurre distributed along the joist in the same manner as those due to increase load. Additional evidence of this was obtained from the tests on the full

cased beams, and it is in accordance with the established fact that creep proportional to stress.

The axial and maximum flexural stresses in the joists of the fully cased beams Nos. T3 to T5, together with the theoretical values including d excluding the concrete in tension, are shown in *Figs. 21* to *23* (pp. 94-), and the relation between central deflexion and maximum flexural stress the steel in *Fig. 24* (p. 96).

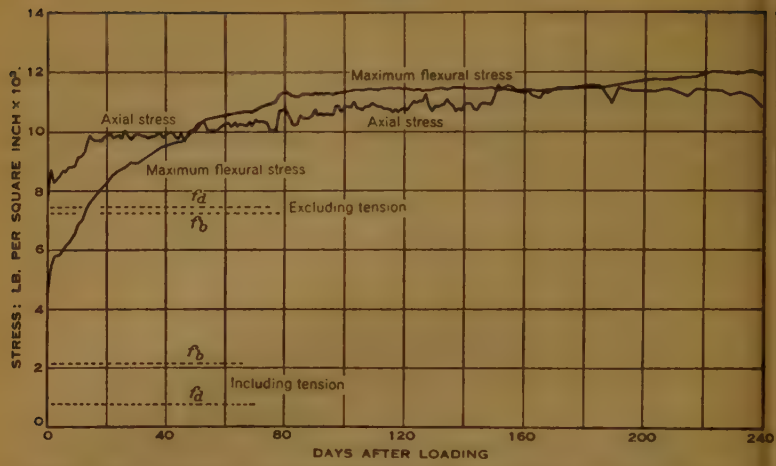
Fig. 20.



Relation Between the Central Deflexion of the Beam and the Maximum Flexural Stress in the Joist for Beams Nos. T9A and T11.

The beam tested in the constant temperature room (No. T3) showed ir-cracks on loading. The flexural stress in the steel was between the eoretical values, but the direct stress was high in relation to the bending ess, indicating some anchorage-effect. The strains in the concrete at e top and the bottom of the slab were measured in this beam, and are own together with the strains in the steel in *Fig. 25* (p. 97). The outward ovement of the steel relative to the concrete at the beginning of the test o indicates anchorage due to the end connexions. As the test continued e relative movement decreased, but full composite action was never

Fig. 21.

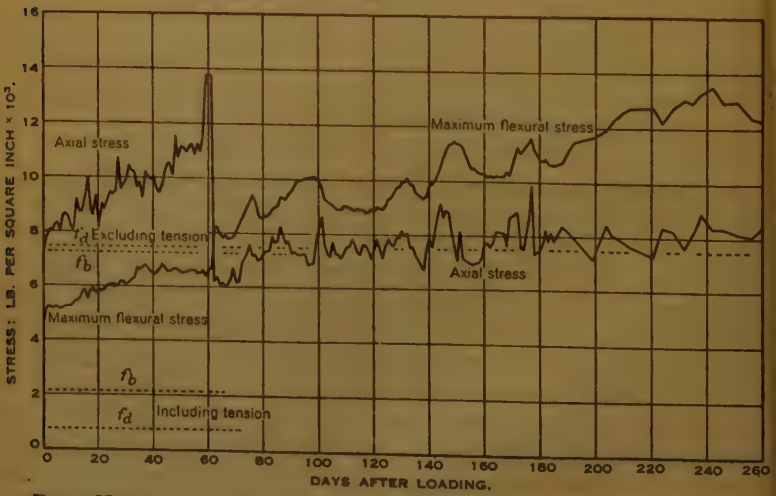


BEAM NO. T3: AXIAL AND MAXIMUM FLEXURAL STRESSES IN THE JOIST.

attained. These effects are again shown by *Fig. 24* (p. 96). The experi mental points for the early part of the test are below the theoretical line but those for the later part are almost in correspondence with it, showing some development of composite action counteracting the effect of anchorage

The direct stress in beam No. T4 (tested out of doors) on loading was too high in relation to the flexural stress for complete composite action indicating some anchorage-effect. This is borne out by *Fig. 24* (p. 96)

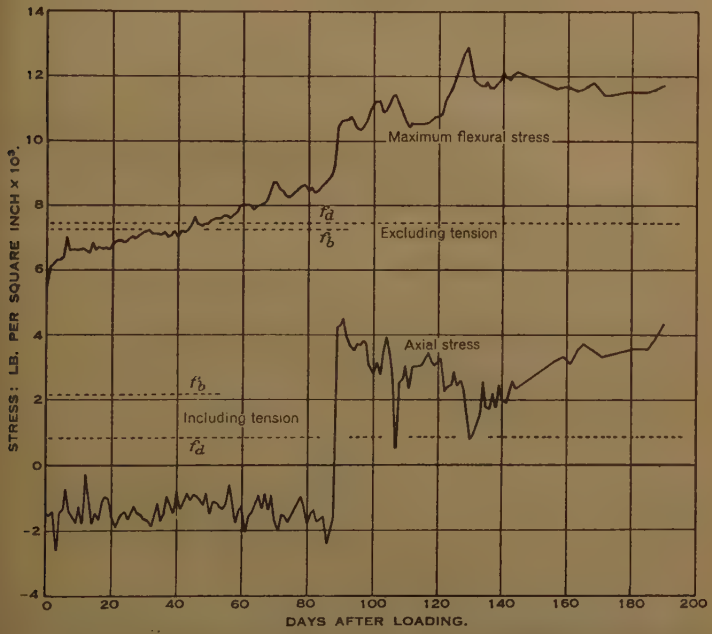
Fig. 22.



BEAM NO. T4: AXIAL AND MAXIMUM FLEXURAL STRESSES IN THE JOIST.

For the first 58 days the anchorage and bond appear to have increased. During this time the weather was cold with considerable wet periods. On the 59th day the temperature was higher and the humidity low, and the temperature of the joist was noticeably lower than that of the air. A large increase in direct stress unaccompanied by an increase in bending stress occurred, due to differential expansion. After this the temperature dropped, and between the 61st and 62nd days there was a remarkable decrease of direct stress accompanied by a smaller increase of bending stress, without a marked increase in deflexion. The corresponding points

Fig. 23.



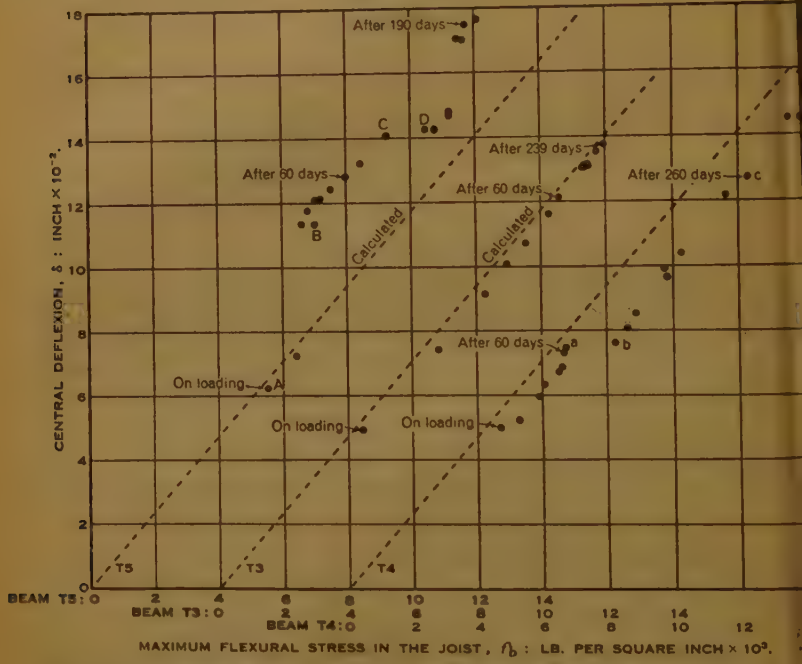
Beam No. T5: Axial and Maximum Flexural Stresses in the Joist.

in *Fig. 24* are a and b, indicating an increase in the proportion of the direct stress due to anchorage. The direct stress itself, however, decreased, and part of the decrease was probably due to differential temperature-effects; but the small increase in flexural stress, together with the position of the points a and b, show that the major part of the decrease was due to breakdown of bond, leaving the axial force to be supplied by the end connexions. After the 62nd day the part bc in *Fig. 24* indicates a continued development of composite action as in beam No. T3, since c is proportionally nearer to the calculated line than b.

Beam No. T5 behaved in a remarkable manner on loading. No hair-cracks could be detected, and there was a compressive axial stress in the

joist after loading. This is difficult to explain, but a compressive axi force was also obtained in the course of the experiments on beam No. T The point A in *Fig. 24* is practically on the theoretical line. A compressiv axial force acting without bond-resistance would bring A above th theoretical line. It is therefore probable that there was some bon resistance. After 5 days there was a sudden increase of deflexio accompanied by only a small variation in the steel stresses. The corr

Fig. 24.



Relation between the Central Deflexion of the Beam and the Maximum Flexural Stress in the Joist for Beams Nos. T3, T4 and T5.

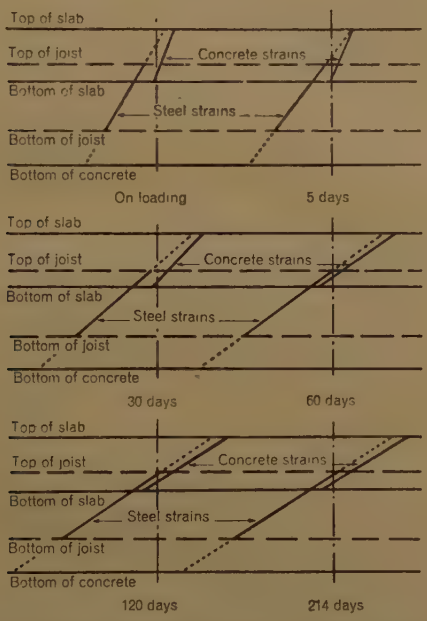
sponding point B in *Fig. 24* shows that slip had taken place. Fine crac were noticed on the 20th day, but there was no marked change in t behaviour of the beam until between the 88th and 89th days, when the was a very large increase of direct stress, changing it from compressio to tension, and a fairly large increase in the maximum flexural stress. T corresponding change in *Fig. 24* was from C to D, indicating a sudd development of bond-resistance. After further irregularities of behavio the beam appears to have settled down to normal conditions. At the e of 2½ years the direct stress had reached 10,000 lb. per square inch and t bending stress 13,200 lb. per square inch.

Although a full explanation of the effects of exposure to the weath

annot be given owing to the number of factors involved, the above nalysis shows that the major irregularities in the behaviour of the beams ere due to interchanges between the action of the end connexions and ne bond. Further evidence of this was obtained from the tests on beams os. T6 to T8, which were also used to investigate the effect of rain.

These beams were subjected for a time to periodic wetting and drying. he maximum flexural stresses in the steel are plotted, together with the entral deflexions, in *Fig. 26* (p. 98), the wet and dry periods being indicated

Fig. 25.



The Relative Movement of the Steel and Concrete in Beam No. T3.

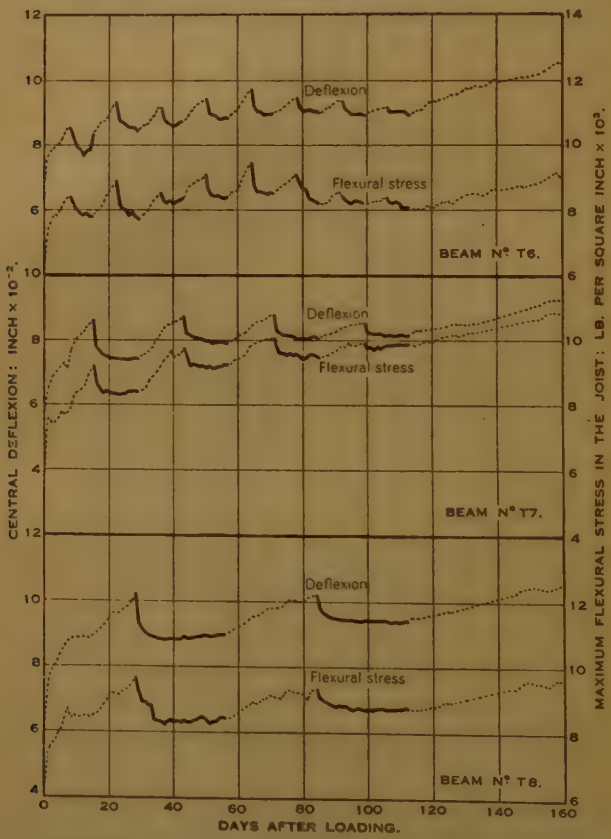
y full and broken lines respectively. In every case there was a rapid ecrease of flexural stress and central deflexion on wetting, followed, in me cases, by a slight rise towards the end of the wet period. This grees with the behaviour of beams Nos. T4 and T5 during rainy periods *Figs. 16* and *22*, *17* and *23*).

The relation between the central deflexion and the maximum flexural ress for beam No. T7 is shown in *Fig. 27* (p. 99). The progressive ecrease in $\frac{\delta}{f_b}$ during the time that the beam was subjected to periodic etting indicates that the effect of the wetting was to increase the pro- ortion of the total axial force which was due to anchorage. The ratios

for beams Nos. T6 and T8 are complicated owing to other actions, such as slip, being superimposed upon this effect.

The axial stresses are shown in *Fig. 28*. The irregularities due to fluctuations in temperature and humidity, and to slip, make it difficult to observe the effect of the wetting, but there is some indication that

Fig. 26.

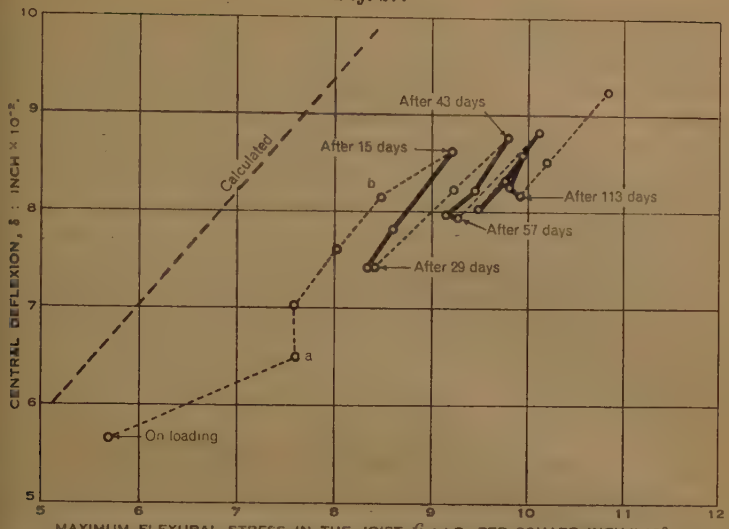


CENTRAL DEFLEXIONS AND MAXIMUM FLEXURAL STRESSES IN THE STEEL IN THE BEAMS SUBJECTED TO PERIODIC WETTING.

causes an increase in the axial stress. It is therefore probable that the increase in the proportion of the total axial force which was due to anchorage was caused by an actual increase in anchorage rather than a decrease in bond.

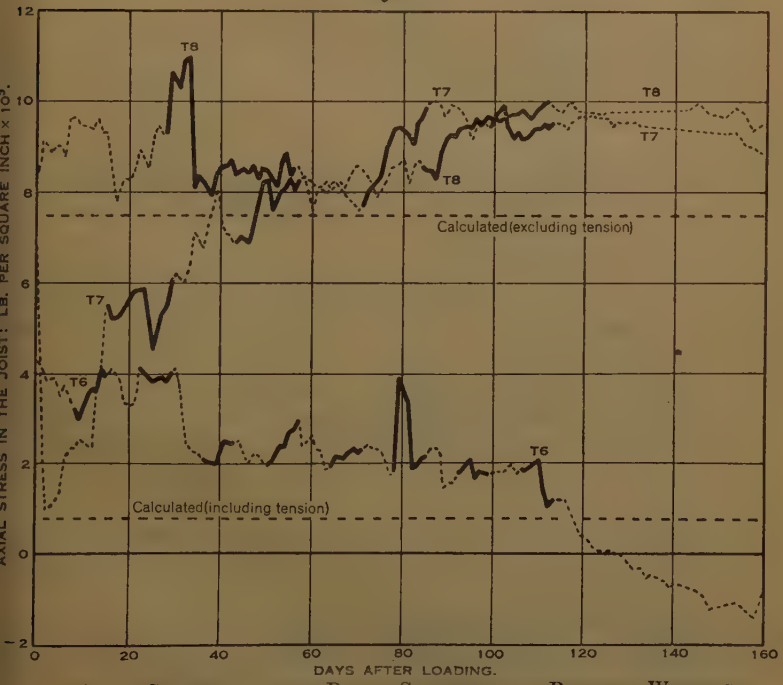
Sudden changes occurred in the axial stresses in these beams, as in those tested out of doors. For example, there was a decrease of axial stress in beam No. T7 during the first 2 days after loading, without an

Fig. 27.



RELATION BETWEEN THE CENTRAL DEFLEXION OF THE BEAM AND THE MAXIMUM FLEXURAL STRESS IN THE JOIST.

Fig. 28.



AXIAL STRESSES IN THE BEAMS SUBJECTED TO PERIODIC WETTING.

abnormal increase in deflexion. This was similar to the behaviour o beam No. T4 on the 61st day.

After this sudden decrease the axial stress rose; *ab* (*Fig. 27*), show that this was mainly due to the development of composite action. I beam No. T8, also, after the decrease of anchorage on the 33rd day composite action appears to have been established. These changes are o the same type as the sudden increase of stress in beam No. T5.

The axial stress in beam No. T6 was low at the beginning. Althoug it increased during some of the wet periods, several slips occurred simila to the one in beam No. T8, and it fell to about 1,000 lb. per square inc on the 113th day after loading. After this the beam was allowed to dr out, and the shrinkage of the concrete caused the axial force to becom compressive. As in beam No. T5, the bond-resistance was evidentl deficient.

Shrinkage Stresses and Yield.

Up to the present only the deformations after application of the load have been considered. Actually there are strains in the steel and concret before the beam is loaded, owing to the shrinkage which has taken place although they have been decreased somewhat by creep [1].

Shrinkage phenomena in plain and reinforced concrete have been the subject of many investigations. The most extensive experiments in Grea Britain were carried out at the Building Research Station, and the result are given in the publications of the Department of Scientific and Industria Research and elsewhere. The amount of shrinkage in a given tim depends upon the cement, mix, size of specimen, conditions of the atmo sphere, and other factors. A factor which is not generally considered i the variation of the shrinkage from the top to the bottom of a specimen This was shown by M. H. Bakir in the course of the experiments a Birmingham mentioned above, and confirmed later by A. A. Khalek [2].

In a simply-reinforced beam the unsymmetrical placing of the stee rods causes bending as well as direct stresses when shrinkage take place. The stresses can be calculated for a given free shrinkage of th concrete, assuming the rods to have no resistance to bending [3]. Th calculation is more complicated for a composite beam since the stiffnes of the joist cannot be ignored, and an additional equation is required. Th formulas, taking into account the effect of uneven shrinkage, are given i Appendix III (pp. 112 *et seq.*).

Measurements of the deflexion and steel strains in beam No. T2 wer taken before the beam was loaded (*Figs. 29*). During the period of mois

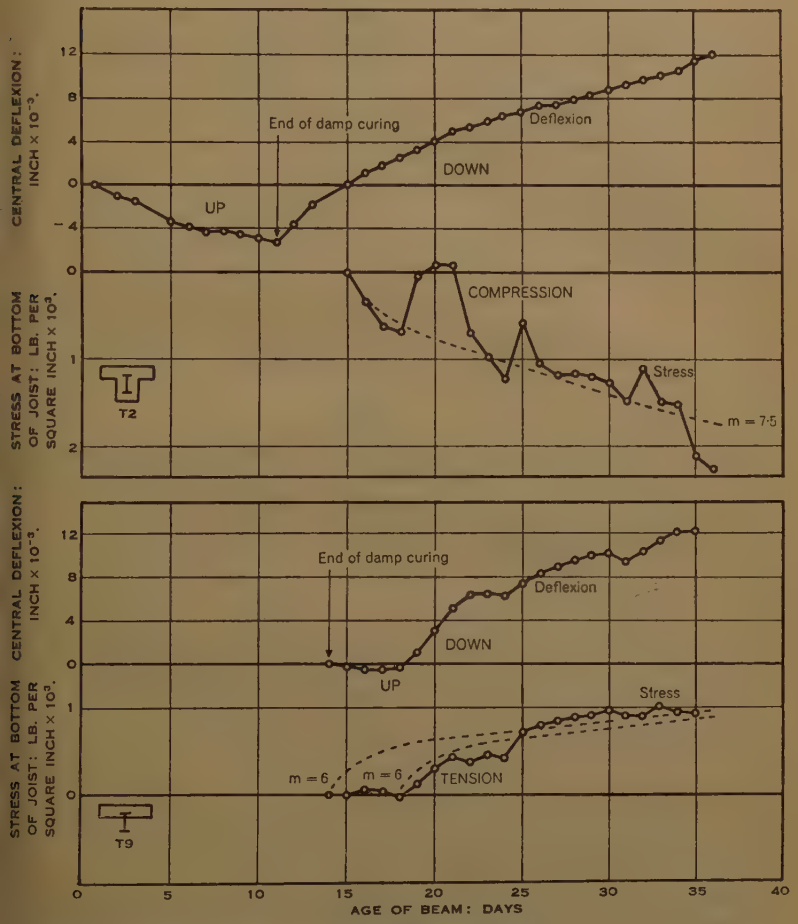
[1] W. H. Glanville and F. G. Thomas, *loc. cit.*, see p. 89.

[2] H. W. Coultas, "The Effect of the Coarse Aggregate and other Factors on th Properties of Concrete." Journal Inst. Struct.E., vol. 14 (1936), p. 131.

[3] Oscar Faber, *loc. cit.*, see p. 87.

uring the deflexion was upward at mid-span, but began to diminish when he damping ceased, reached zero after 4 days and then became downward. Readings of the extensometers were taken from the time when the deflexion

Figs. 29.



Central Deflexions and Stresses at the Bottom of the Joist in Unloaded Beams.

eached zero, since, presumably, before this the deformations would be due o recovery from the previous expansion.

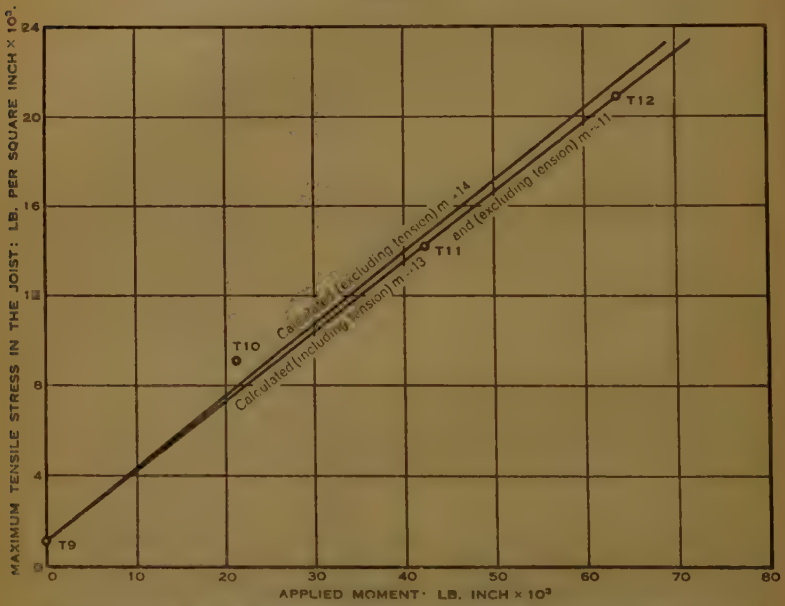
A similar test was carried out on one of the partially encased joists No. T9). The curing lasted for 14 days, and the readings of deflexion nd steel strain were commenced immediately after the damping ceased.

Values for the shrinkage of a concrete prism made from the same

cement and aggregates as beams Nos. T2 and T9 can be obtained from the curves given in the Paper by H. W. Coultas to which reference has already been made, making allowance for the different period of curing. Using these values the shrinkage stresses may be calculated, and the theoretical shrinkage curves are shown as dotted lines in the figure.

Although the development of compressive stress at the lower surface of the fully encased joist was very irregular its general trend was in accordance with the theoretical curve. The records of humidity and temperature showed that the major irregularities were due to changes in

Fig. 30.



Application of an Effective Modular Ratio to the Experimental Results for the Partially Encased Joists.

these, the effects being similar to those observed in the experiments described earlier.

The development of shrinkage stress in the partially encased joist was delayed at the beginning owing to the high humidity of the air, and it is probable that the shrinkage curve should start at 18 days. This curve is also shown in the figure. Both curves are in fair agreement with the experimental results after 25 days. In this beam there would be a certain amount of creep due to the weight of the beam, which would increase the tension in the lower flange.

The variation of shrinkage across the section of the beam has an important effect upon the shrinkage stresses. For example, the calculated

shrinkage stress at the lower surface of the joist in No. T9 would be negligible if the shrinkage were uniform.

The observed stresses in the lower flanges of the partially encased joists Nos. T10 to T12, 6 months after loading, are plotted in *Fig. 30* against the applied moment. The stress in the unloaded beam, No. T9, is also plotted, this being the difference between the observed shrinkage stress of 880 lb. per square inch when the beam was 28 days old (*Figs. 29*) and of 2,040 lb. per square inch 6 months later. This value was taken since beams Nos. T10 to T12 were loaded when 28 days old. It has been established that the creep of concrete is proportional to the stress [1] up to stresses higher than those reached in these beams. Thus, if the shrinkage stress were the same in all the beams at the same age, the points in the figure should lie upon a straight line. This condition is fulfilled by the points for beams Nos. T9, T11, and T12. From the slope of the line the "effective" modular ratio [2] may be determined. It is 13 if the concrete in tension is included and 11 if it is excluded.

Further Experiments and Direct-Loading Tests to Failure.

Large residual stresses remained in beams Nos. T5 to T8 when they were unloaded, the stresses in the lower flanges of the joists only decreasing by about 7,000 lb. per square inch. The flexural stresses in the joists diminished gradually with time, but the changes in direct stress were irregular. On reloading, the beams returned to approximately the same condition as before unloading.

Three of the beams were subjected to twice the working load for about a month and showed no sign of yield of the steel. One beam was afterwards loaded to 2·9 times the working load, which overstrained the steel and produced fine cracks about $\frac{1}{2}$ inch long in the vertical faces of the slab. Noticeable creep of the steel ceased after a few hours, and there was only a slight extension of the cracks in the concrete after 32 days under this load.

All the specimens except Nos. T4 and T8 were tested to failure, and the results are given in Table IV (p. 104), together with the results of direct-loading tests to failure on two beams Nos. T1*a* and T9*a* similar to Nos. T1 to T8 and Nos. T9 to T12 respectively. The approximate moment at which yield of the steel occurred was determined, and is given in the Table. The failure of all the beams was gradual, and the central deflexion exceeded 1 inch. After beam No. T5 failed the machine was balanced again, and the beam would still carry 89 per cent. of the failing load. The general behaviour of the beams was similar to that in the tests described in Part I.

[1] W. H. Glanville, "Creep or Flow of Concrete under Load." Building Research Technical Paper No. 12.

[2] W. H. Glanville and F. G. Thomas, "Creep of Concrete under Load." Journal Inst. Strut.E., vol. 11 (1933), p. 67.

TABLE IV.—RESULTS OF THE TESTS TO FAILURE.

Beam No.	Type.	Approximate age at test to destruction.	Approximate total moment at yield of steel [2]: lb.-inches × 10^3.	Total moment at failure [2]: lb.-inches × 10^3.	Approximate load-factor on yield.	Load-factor on failure.	Remarks.
T1*a*	Fully encased. Mix (*a*)	28 days.	119	200	2·40 [1]	4·02	
T1	,, ,,	3½ years.	123	219	2·47	4·40	
T2	,, ,,	3½ years.	105	231	2·11	4·65	Failure due to crushing of the concrete. No evidence of slip.
T3	,, ,,	4 years.	115	231	2·31	4·65	
T5	,, ,,	2½ years.	107	210	2·15	4·22	
T6	,, ,,	1 year 69 days.	156	225	3·13	4·52	
T7	,, ,,	343 days.	112	211	2·25	4·24	
T9*a*	Partially encased. Mix (*c*)	28 days.	134	194	2·38 [1]	3·44	
T9	,, ,,	2½ years.	146	218	2·58	3·86	
T10	,, ,,	2½ years.	118	196	2·10	3·48	Failure due to slip of end connexion following yield of steel and cracking of slab.
T11	,, ,,	2½ years.	109	197	1·93	3·50	
T12	,, ,,	2½ years.	105	217	1·86	3·85	
T13	,, ,, (No end connexions.)	2½ years.	118	155	2·10	2·75	Joist slipped inwards after yield of steel and cracking of slab.

[1] The load-factor based upon the prism-modulus is 2·14 for beam No. T1*a* and 1·93 for beam No. T9*a*.

The load-factors given are based upon the calculated working moments neglecting tension in the concrete, using a working stress of 18,000 lb. per square inch in the steel and $m=18$ for mix (*a*) and 14 for mix (*c*). The dead loads were allowed for in the calculation for the fully encased beams.

[2] Including the moment due to the weight of the beam.

The fully encased beams which had been loaded before had hair-cracks in the stem before the tests were carried out. There was no sudden development of these when the steel yielded, but they extended gradually, reaching the vertical face of the slab at moments varying from 25,000 to 30,000 lb.-inches above the yield.

No cracks appeared in the partially encased beams (Nos. T9*a* to T12) until yield of the steel took place, when hair-cracks developed under the loading points. Finally, cracks appeared in the vertical pieces at the ends of the beam, indicating give of the end connexions. If these had been stronger it is probable that the beams would have withstood higher loads and have failed by crushing of the concrete. There were no cracks in the end pieces of the fully encased beams, and they gave higher load-factors on failure.

The importance of anchorage was shown in Part I and is further illustrated by the results of the test on beam No. T13, which had no end connexions. The joist slipped in after yield of the steel and cracking of the slab, and failure occurred at a much lower load than in any of the other beams.

The load-factors on yield and ultimate strength given in the Table are based upon the computed working loads, using modular ratios of 18 for beams Nos. T1*a* to T8 and 14 for beams Nos. T9*a* to T13, and a working stress of 18,000 lb. per square inch in the steel. These values are in accordance with the Recommendations for a Code of Practice for Reinforced Concrete. The shuttering was suspended from beams Nos. T1*a* to T8 when the concrete was poured. The calculated bending moment in the steel due to the weight of concrete was 3,700 lb.-inches and the maximum tensile stress 2,700 lb. per square inch. Thus, in the calculation of the working moment for applied loads, the steel stress taken was 15,300 lb. per square inch. This gives an allowable applied moment of 46,100 lb.-inches. Adding the dead-load moment the working moment becomes 49,800 lb.-inches. The weight of the shuttering was ignored, since it was taken off before the beam was loaded. Beams Nos. T9*a* to T13 were supported during pouring, and the above considerations do not apply. The working moment was 56,400 lb.-inches.

The values given for the moment at yield are only approximate, since yield occurs gradually and its effect is sometimes difficult to separate from the action of other factors. In general, however, the load-factor on yield decreased after a period of sustained loading, and this was more marked in the partially encased beams. On the other hand the load-factor was greater in beam No. T6 than in any of the other beams. This was due to the beam having previously carried a load which overstressed the steel for a considerable period.

The average value of the load-factor on yield for the fully encased beams, omitting beams Nos. T1 and T6, was 2·20, and it was 1·96 for the partially encased beams Nos. T10 to T12. It was higher for the beams

Nos. T1*a* and T9*a*, tested when 28 days old. In obtaining these figures, however, the effective modular ratio was used, which results in the steel being understressed, and the concrete overstressed, when the beam is first loaded to the working moment. The moment at which the stress in the steel would reach 18,000 lb. per square inch may be determined by using the prism-modulus, and, for purposes of comparison with the experiments under sustained loading, this should be taken as the working moment in determining the load-factor for beams Nos. T1*a* and T9*a*. The concrete in tension should also be included in beam No. T9*a*, since there was no cracking until the steel had yielded. The load-factors then become 2·14 and 1·93 for beams Nos. T1*a* and T9*a* respectively. These values are close to the averages for the beams which had been under sustained loading, which shows that the increased ratios given by the Code of Practice cover sufficiently the effect of sustained loads on the steel for these beams.

The figures for the load-factors on breaking, given in Table IV, disclose the important fact that the factors were greater for all the beams which had carried sustained loads than for a similar beam loaded to failure when 28 days old. Further, the range of the load-factors, both for the fully and partially encased beams, was small in spite of the dissimilar histories of the individual beams.

The results also show that, as in the tests on the beams with anchorages described in Part I, there was a large reserve strength after the steel had commenced to yield. A beam, No. T13, similar to beams Nos. T9 to T12, but without end connexions, was subjected to the same period of sustained loading as these beams, and afterwards tested to destruction. The results are given in Table IV. In the tests on beams without anchorage described in Part I, failure by slip occurred before the steel yielded. In this beam the bond stress was low and slip only took place after yield of the steel had commenced. There was, however, only a small reserve of strength after yield.

The increase in the stresses in the steel during a period of sustained loading, and the resultant lowering of the load-factor on yield, is accompanied by a decrease in the compressive stresses in the concrete which increases the reserve strength after yield. In these tests, the two effects appear to have counterbalanced one another, with the result that the load-factor on breaking was very little, if at all, different from the factor which would have been obtained if the beam had remained unloaded and under normal conditions of storage. It is probable that this counterbalance always occurs when failure is due to crushing of the concrete. If so, it would follow that the load-factor of a beam having efficient anchorages or end connexions would always be greater, after prolonged loading, than that of a similar beam tested when 28 days old. A direct-loading test would therefore be sufficient for estimating the load-factor on failure. How far this applies to beams which fail in other manners is

unknown, except that it holds for beams Nos. T9*a* to T12 which failed by slip of the end connexions.

Summary and Conclusions. Part II.

(1) The increase of deflexion of the fully encased beams under sustained loading was of the same order as in reinforced-concrete beams, when the tests were carried out in the laboratory, and the maximum tensile stress in the steel increased in the same manner as the deflexion.

(2) The experiments on both reinforced-concrete and composite beams showed that normal changes of humidity of the atmosphere produce fluctuations of deflexion in correspondence with them, increase of humidity decreasing the deflexion and vice versa. Changes of temperature produce similar effects. These are superimposed on the normal time-effects.

During periods of high humidity the general rate of increase of deflexion is much less than during periods of low humidity.

(3) Exposure to weather during test caused the changes in deflexion to become much more irregular, and remarkable variations occurred in the maximum tensile stress in the steel. The variations in deflexion and stress, which were sometimes very rapid and of large magnitude, were not always in accordance with one another.

(4) Consideration of the axial and flexural stresses in the joists in conjunction with the deflexions showed that there are interchanges between the action of the end connexions and the bond between the joist and the concrete. These become complicated when the beam is exposed to weather, leading to the sudden large changes in stress and deflexion, not always in accordance with one another, referred to above.

(5) The major irregularities in the behaviour of the joists under load were due to changes in the axial force. The bending moment in the joists increased in a fairly regular manner except for the fluctuations due to changes of humidity and temperature.

(6) There is some evidence that the increased flexural stresses in the steel due to creep of the concrete are, under normal conditions, distributed along the joist in the same manner as those due to an increase of load.

(7) Wetting the beam produces a rapid decrease in the flexural stress in the steel and in the deflexion. When the wetting is prolonged this is followed by an increase in these, but at much less than the normal rate.

(8) The shrinkage stresses in the steel were determined in a partially encased and a fully encased joist, and were found to be of the same order as those obtained theoretically by the method given in Appendix III (pp. 112 *et seq.*).

The variation of shrinkage across the section of the beam has an important effect upon the shrinkage stresses.

(9) Large residual stresses remained in the beams subjected to pro-

longed loading after the load was removed. On reloading, the beams returned to approximately the same condition as before unloading.

(10) Three fully encased beams, which had carried the working load for a considerable period, were subjected to twice the working load for about a month and showed no sign of yield of the steel. The load on one of these beams was afterwards increased until the steel was overstrained. Noticeable creep of the steel ceased after a few hours, and there was only a slight extension of the cracks in the concrete after 32 days.

(11) In none of the tests did the cracks extend into the slab until after the steel had yielded.

(12) The load-factor on yield of the steel (load at yield/working load) after a long period of sustained loading was less than that of a similar unloaded beam 28 days old, but the use of the effective modular ratios given in the Recommendations for a Code of Practice for Reinforced Concrete was sufficient to cover the decrease.

(13) The load-factors on failure of all the beams which had carried sustained loads were greater, in spite of the diverse histories of the beams, than that of a similar beam loaded to failure when 28 days old.

(14) There is a large reserve of strength after yield of the steel commences if the end connexions give sufficient anchorage, and, under sustained loading, this reserve increases with time, owing to the decrease in the compressive stresses in the concrete. In the present tests this effect counterbalanced the lowering of the moment at which yield of the steel occurred.

The large fluctuations and sudden changes of stress in the beams tested out of doors may, at first sight, lead to some doubt as to the safety of composite beams, but there is no reason to suppose that reinforced-concrete beams behave in a less erratic manner under similar conditions. The high values of the load-factors on failure are reassuring, and the reserve of strength after yield of the steel commences compares favourably with the breakdown of an ordinary reinforced-concrete beam under similar circumstances. It has been suggested by some writers that higher stresses might be permitted in composite beams than in reinforced-concrete beams. This is borne out to some extent by the present tests, but consideration must be given to the decrease in the load-factor on yield of the steel which may sometimes be produced by anchorages.[1] Further investigations on the arrangement of anchorages are desirable.

Acknowledgements.

The Authors wish to express their gratitude to the Department of Scientific and Industrial Research for a grant which made it possible for

[1] Cf. Part I, p. 80.

them to carry out a great part of the work described in the Paper. They are also indebted to Mr. K. Summerfield, M.Sc., a graduate student, who carried out many of the tests on partially encased joists.

The Paper is accompanied by forty-six diagrams, from some of which Plate 1 and the Figures in the text have been prepared, and by three photographs and the following Appendixes.

[APPENDIXES.

APPENDIX I.

Notation.

d_1 = the total depth of the beam.
d = the depth to the bottom of the joist.
c = the depth of concrete above the top flange of the joist.
b = the width of a rectangular beam.
b_2 = the width of the slab of a T-beam.
b_3 = the width of the stem of a T-beam.
d_2 = the depth of the slab of a T-beam.
A_s = the area of cross section of the joist.
A_c = the area of cross section of the concrete.
A = the area of cross section of the beam with the steel replaced by an equal area of concrete.
kd = the depth of the neutral axis.
y_s = the depth to the centre of gravity of the joist.
y_c = the depth to the centre of gravity of the concrete.
y = the depth to the centre of gravity of the beam with the cross section of the steel replaced by an equal cross section of concrete.
I_e = the moment of inertia about the neutral axis of the area obtained by considering the beam as homogeneous, the cross section of the concrete being replaced by an equivalent cross section of steel.
I_s = the moment of inertia of the joist about an axis through its centroid parallel to the flange.
I_c = the moment of inertia of the cross section of the concrete about an axis through its centroid parallel to the flange of the joist.
I = the moment of inertia of the cross section with the cross section of the steel replaced by an equal cross section of concrete, about an axis through its centroid parallel to the flange of the joist.
E_s = Young's modulus for steel.
E_c = Young's modulus for concrete.
$m = \frac{E_s}{E_c}$ = the modular ratio.
f_s = the fibre stress at the bottom of the joist.
f_{s1} = the fibre stress at the top of the joist.
f_d = the axial stress in the joist.
f_b = the maximum flexural stress in the joist.
f_c = the fibre stress at the top of the concrete.
f_{c1} = the fibre stress at the bottom of the concrete.
f_{dc} = the axial stress in the concrete.
s_b = the bond stress at the upper surface of the joist.
M = the bending moment.
S = the total shearing force over the cross section.
q_s = the free shrinkage at the top of the concrete.
s = the free shrinkage at the bottom of the concrete.

APPENDIX II.

Formulas for Calculating the Position of the Neutral Axis and the Stresses in Composite Beams.

$$f_s = \frac{M(d - kd)}{I_e}.$$

$$f_c = \frac{Mkd}{mI_e}.$$

$$f_d = \frac{M(y_s - kd)}{I_e}.$$

$$f_b = \frac{M(d - y_s)}{I_e}.$$

$$s_b = \frac{Sc(kd - c/2)}{mI_e}.$$

On assumption (1) (including the concrete in tension)

$$kd = \frac{A_c y_c + mA_s y_s}{A_c + mA_s}$$

and

$$I_e = \frac{I_c}{m} + \frac{A_c}{m}(kd - y_c)^2 + I_s + A_s(y_s - kd)^2$$

for beams having any form of cross section.

For fully encased beams these formulas may be stated in a more practical form by substituting the following values for A_c, y_c, and I_c:

$$A_c = A - A_s,$$

$$y_c = \frac{Ay - A_s y_s}{A - A_s},$$

and

$$I_c = I + A(y - y_c)^2 - I_s - A_s(y_s - y_c)^2$$

$$= I - I_s - \frac{AA_s}{A - A_s}(y_s - y)^2.$$

Then

$$kd = \frac{Ay + (m-1)A_s y_s}{A + (m-1)A_s},$$

and

$$I_e = \frac{I}{m} + \frac{A}{m}(kd - y)^2 + \frac{m-1}{m}\{I_s + A_s(y_s - kd)^2\}.$$

For partially encased beams this simplification cannot be made, but in most cases kd and I_e may be obtained to a sufficient degree of approximation by substituting A, y, and I for A_c, y_c, and I_c respectively. (Mohr's approximation.)

On assumption (2) (neglecting the tension in the concrete and using Mohr's approximation),

(*a*) rectangular beam,

$$kd = -\frac{mA_s}{b} + \sqrt{\left(\frac{mA_s}{b}\right)^2 + \frac{2mA_s y_s}{b}},$$

$$I_e = \frac{b(kd)^3}{3m} + I_s + A_s(y_s - kd)^2.$$

(*b*) T-beam.

If the neutral axis is within the slab the formulas for a rectangular beam may be used by substituting b_2 for b.

If the neutral axis is in the stem

$$kd = -\frac{mA_s}{b_3} - \frac{d_2}{b_3}(b_2 - b_3) + \sqrt{\left\{\frac{mA_s}{b_3} + \frac{d_2}{b_3}(b_2 - b_3)\right\}^2 + \frac{2mA_s y_s}{b_3} + \frac{d_2^2}{b_3}(b_2 - b_3)},$$

and

$$I_c = \frac{b_2 d_2^3}{3m} + \frac{b_2 d_2 kd}{m}(kd - d_2) + \frac{b_3}{3m}(kd - d_2)^3 + I_s + A_s(y_s - kd)^2.$$

It is impracticable to give a general formula for beams having any form of cross section.

APPENDIX III.

Shrinkage Stresses in Composite Beams.

It will be assumed that plane cross sections remain plane after shrinkage.

Then

$$kd = \frac{\frac{q-1}{d_1}\{I_c A_s y_s - I_s A_c y_c - A_s A_c y_s y_c (y_s - y_c)\} + qA_c\left\{I_s + \frac{I_c}{m} + A_s y_s (y_s - y_c)\right\}}{\frac{q-1}{d_1}\left\{I_c A_s + I_c \frac{A_c}{m} - A_s A_c y_c (y_s - y_c)\right\} + qA_s A_c (y_s - y_c)},$$

and

$$f_s = \frac{E_c s A_c (kd - d)\left(q - \frac{q-1}{d_1} y_c\right)}{A_s(kd - y_s) + \frac{A_c}{m}(kd - y_c)}$$

for beams having any form of cross section.

For fully encased beams the formulas may be stated in a more practical form by substituting the values of A_c, y_c, and I_c, as in Appendix II.

Then

$$kd = \left[\frac{q-1}{d_1}\{IA_s y_s - I_s A y - A_s A y_s y (y_s - y)\} + q\left\{\frac{(m-1)I_s + I}{m}(A - A_s) + A_s A (y_s - y)\frac{(m-1)y_s + y}{m}\right\}\right] \div \left[\frac{q-1}{d_1}\left\{(I - I_s)\frac{(m-1)A_s + A}{m} - A_s A (y_s - y)\frac{(m-1)y + y_s}{m}\right\} + qA_s A (y_s - y)\right]$$

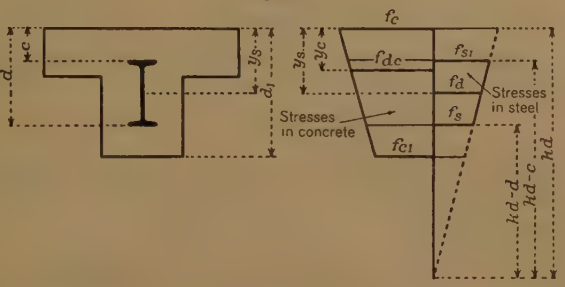
and

$$f_s = \frac{E_c s (kd - d)\left\{q(A - A_s) - \frac{q-1}{d_1}(Ay - A_s y_s)\right\}}{\frac{m-1}{m} A_s (kd - y_s) + \frac{A}{m}(kd - y)}.$$

For partially encased beams Mohr's approximation may be used, A, y, and I being substituted for A_c, y_c, and I_c respectively in the general equations.

The general formulas were derived in the following manner.

Fig. 31.



On the assumption that plane sections remain plane,

$$f_{dc} = f_{c1} + \frac{d_1 - y_c}{d_1}(f_c - f_{c1})$$

$$= f_c - \frac{y_c}{d_1}(f_c - f_{c1}) \quad \ldots\ldots\ldots\ldots \quad (1)$$

and

$$f_{s1} = \frac{kd - c}{kd - d} f_s \quad \ldots\ldots\ldots\ldots \quad (2)$$

Equating the algebraic difference between the steel and concrete strains at the top of the concrete to the free shrinkage.

$$\frac{kd}{kd - d}\frac{f_s}{E_s} + \frac{f_c}{E_c} = qs.$$

Therefore

$$f_c = E_c q s - \frac{kd}{kd - d}\frac{f_s}{m} \quad \ldots\ldots\ldots\ldots \quad (3)$$

Similarly at the bottom of the concrete,

$$f_{c1} = E_c s - \frac{kd - d_1}{kd - d}\frac{f_s}{m} \quad \ldots\ldots\ldots\ldots \quad (4)$$

The algebraic sum of the forces across the section is zero. Therefore

$$\frac{f_{s1} + f_s}{2} A_s = f_{dc} A_c.$$

Using equations (1) to (4) this becomes

$$f_s\left\{\frac{2kd - c - d}{2} A_s + kd\frac{A_c}{m} - y_c\frac{A_c}{m}\right\} = E_c s A_c (kd - d)\left(q - q\frac{1}{d_1}y_c\right).$$

But

$$\frac{c + d}{2} = y_s.$$

Therefore

$$f_s\left\{(kd - y_s)A_s + kd\frac{A_c}{m} - y_c\frac{A_c}{m}\right\} = E_c s A_c (kd - d)\left(q - \frac{q - 1}{d_1}y_c\right) \quad \ldots\ldots \quad (5)$$

The algebraic sum of the moments of the forces over the section is zero. Therefore

$$\frac{\frac{f_{s1}-f_s}{2}I_s}{\frac{d-c}{2}} = \frac{f_c-f_{dc}}{y_c}I_c + \frac{f_s+f_{s1}}{2}A_s(y_s-y_c).$$

Again using equations (1) to (4) this becomes

$$f_s\left\{I_s+\frac{I_c}{m}-\frac{2kd-c-d}{2}A_s(y_s-y_c)\right\} = E_cI_c(kd-d)\frac{q-1}{d_1}$$

or, since

$$\frac{c+d}{2} = y_s,$$

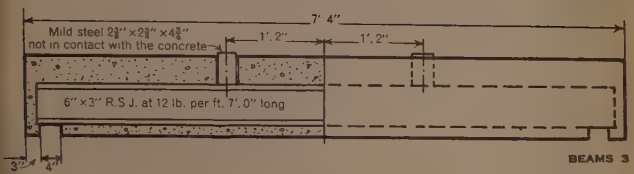
$$f_s\left\{I_s+\frac{I_c}{m}-A_s(kd-y_s)(y_s-y_c)\right\} = E_cI_c(kd-d)\frac{q-1}{d_1} \quad . \quad . \quad (6)$$

Dividing equation (5) by (6)

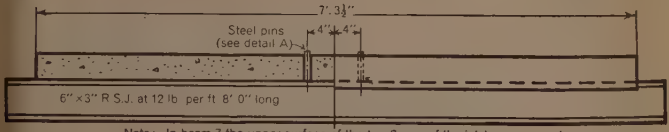
$$kd\left[\frac{q-1}{d_1}\left\{I_cA_s+I_c\frac{A_c}{m}-A_sA_cy_c(y_s-y_c)\right\}+qA_sA_c(y_s-y_c)\right]$$

$$=\frac{q-1}{d_1}\{I_cA_sy_s-I_sA_cy_c-A_sA_cy_sy_c(y_s-y_c)\}+qA_c\left\{I_s+\frac{I_c}{m}+A_sy_s(y_s-y_c)\right\}.$$

BEAMS 1 AND 2.

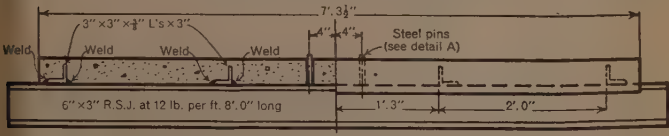


BEAMS 3 AND 4.

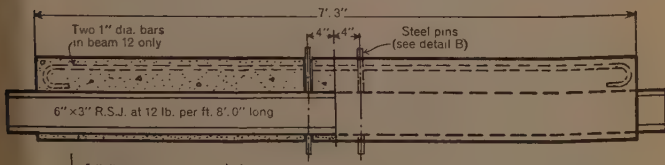


Note:—In beam 7 the upper surface of the top flange of the joist was covered with oiled paper before the concrete was poured.

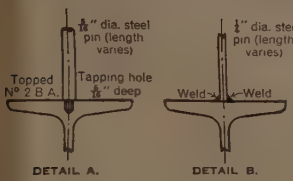
BEAMS 5 AND 7.



BEAM 6.



BEAMS 8, 9 AND 12.



DETAIL A. DETAIL B.

(Scale one quarter full size)

Inches 12 9 6 3

DETAILS

WILLIAM CLOWES & SONS, LIMITED: LONDON.

Paper No. 5161.

"The Resistance to Flow of Water along a Tortuous Stretch of River and in a Scale Model of the Same."

By Jack Allen, M.Sc., Assoc. M. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*)[1]

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Introduction.

In a previous Paper by the Author[2] experiments have been described which led to the conclusion that in straight rectangular open channels, the lower critical Reynolds number, vm/ν, is approximately 1,400 and independent of the roughness of the walls and bed within the range of roughness then investigated. In the same Paper, the problem of similarity in straight open channels was also considered, and suggestions made as to the roughness required for a model relative to its prototype.

The resistance to flow along irregularly winding river-channels contains other complications, and very little is known about such resistance. Thus, the proportionate effect of skin-friction in relation to the loss of energy caused by the bends and sudden changes of cross section clearly depends upon the particular river under consideration: the nature of the material of its bed and banks and the abruptness or otherwise of its bends. Calculations are rendered extremely hazardous, except in the

[1] Correspondence on this Paper can be accepted until the 15th May, 1939, and will be published in the Institution Journal for October 1939.—Sec. Inst. C.E.

[2] "Streamline and Turbulent Flow in Open Channels." Phil. Mag., vol. xvii (Seventh Series), (1934), pp. 1081 *et seq.*

simplest examples of comparatively straight and uniform rivers. In view of this, the tendency has grown within recent years to study river-problems with the aid of scale models.

The question of " scale-effect " in such models then assumes primary importance. In relation to hydraulic resistance, this " scale-effect " is essentially influenced by :

(1) the manner of the motion involved, that is, whether it is stream-line or turbulent,
(2) the textural roughness of the model and its prototype,
(3) the effect of shape, both in plan and in cross section.

It is frequently assumed that of these three influences, the most important and the most difficult to simulate in a model, is the degree of roughness. The present investigation, however, indicates that in the wide class of rivers containing more or less abrupt changes of curvature, the resistance offered by these bends may be at least equal to that caused by textural roughness. Further, it now appears that it is practicable to operate a model of such a river at such low values of Reynolds number as would hitherto have been considered certain to result in a marked " scale-effect " due to viscosity.

Model of River Mersey.

A scale model has been constructed in the Whitworth Engineering Laboratory of Manchester University, under the direction of Professor A. H. Gibson, D.Sc., LL.D., M. Inst. C.E., to study problems concerning certain works of improvement proposed for the non-tidal river Mersey. The investigation was carried out at the request of the Mersey and Irwell Catchment Board, but the opportunity has been taken to make experiments of a more general scientific and engineering interest on the model. The results of these tests, together with observations made on the natural river [1], provide the basis of the present Paper.

The model was made to the scales of 1 to 800 horizontal and 1 to 120 vertical.

There is, consequently, a vertical distortion in the ratio of 6·67 to 1. The scale of velocities, based on the square root of the vertical scale, is 1 to 10·96, and the scale of rate of discharge (volume per unit time) is $1 : (800 \times 120 \times \sqrt{120})$, or $1 : 1{\cdot}05 \times 10^6$.

The portion of river under investigation is shown in *Fig. 1*. Measured along the centre-line of the river, its length is 8·30 miles, or 2·25 times the

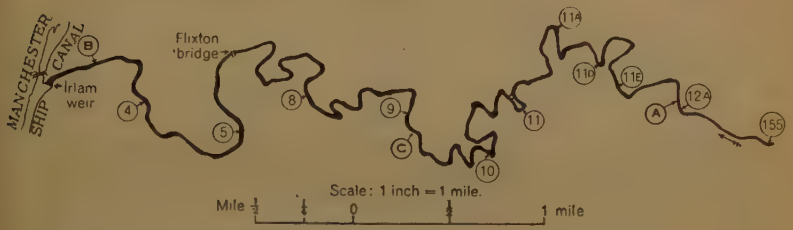
[1] The observations on the natural river were made by the Mersey and Irwell Catchment Board, to whose Chief Engineer, Mr. S. Pearson, M. Inst. C.E., and his late assistant, Mr. J. H. Brooks, B. Eng., Assoc. M. Inst. C.E., the Author is greatly indebted.

shortest distance between the two ends. At the lower end, the river discharges over Irlam weir into the Manchester ship-canal. This weir was accordingly reproduced in the form of a block of teak in the model, using the vertical scale of 1 to 120 for its height and for its dimension parallel to the direction of flow.

It will be seen, from *Fig. 1*, that the river contains no fewer than twenty-eight major bends. Estimates of the rate of flow in the natural river are based upon the recorded head over Irlam weir, which has been calibrated partly by means of current-meter observations and also by means of a 1 : 50 scale model of the weir and its approach channel.

The river model was made of an average mixture of 1 part by volume of Portland cement to 5 parts of fine sand (mean diameter 0·009 inch) and, as will be explained at a later stage, its bed was finally roughened by a sparse sprinkling of coarse sand. The model, thus moulded, was enclosed in a wooden casing whose upper edge was carefully levelled.

Fig. 1.



PLAN OF RIVER MERSEY.

Water is fed to the upper end of the model through one of a selection of calibrated orifices screwed into the bottom of a cylindrical vessel open at the top and fitted with a gauge-glass for measurement of head. The stream thus provided enters the moulded part through screens of perforated zinc, and, as a check, the discharge has always been measured by direct collection at the downstream end of the model.

To compensate for slight variations in the top level of the wooden sides of the model, which acts as a datum corresponding to + 90·00 O.D., readings of all gauges on static water-surface levels were frequently taken, and the zero of each relative to the rest thus derived, while the fixed top edge in the immediate vicinity of Irlam weir was treated as of constant level + 90·00 O.D.

For the measurement of water-surface levels, the general method adopted has been that of pointer-gauges rigidly fastened to strips of angle-iron. These gauges are equipped with verniers enabling readings to be taken directly to $\frac{1}{100}$ inch (equivalent to $\frac{1}{10}$ foot in nature), but estimates could be made with considerable accuracy to one-quarter of this magnitude.

Relevant Facts concerning the Mersey itself.

At station 12A (*Fig. 1*), the bed-level of the river is + 51·00 O.D.; in the vicinity of Irlam weir, some 7·5 miles below, it is + 26·00. Other important details are summarized in the following Tables:

Table I.

Station	11A	10A	10	8	5
Bed-level: feet above L.O.D. .	43	40	39	33	26

Table II.

Discharge: cusecs.	Station A to Station B*.			
	Average depth: feet.	Average area: square feet.	Average value of m †: feet.	Fall in water-surface: feet.
350	5·4	255	3·51	20·0
2,550	12·3	710	7·56	23·9
7,000	18·4	1,370	10·7	29·0

* AB = 39,600 feet.
† m = hydraulic mean depth.

The bed-levels quoted above are those of the lowest point in the cross section. Correspondingly, the water-depths are measured from the water-surface to the deepest point of the river-bed vertically below the point of measurement of the water-level.

For the purpose of estimating the average areas and hydraulic mean depths, twenty-four approximately uniformly spaced cross sections of the river have been considered.

The data given above may now be used to provide an estimate of the proportion of "textural resistance" to total loss of head.

Calling δh the loss of head, or drop in water-surface between stations A and B,

$$\delta h = \frac{flv^2}{2gm} + \frac{v_A^2 - v_B^2}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where v denotes the mean velocity, m the average hydraulic mean depth, v_A the average velocity at station A, and v_B the average velocity at station B.

Actually, the kinetic energy term, $(v_A^2 - v_B^2/2g)$, amounts only to —0·06, — 0·25, and + 0·25 feet for discharges of 350, 2,550 and 7,000 cusecs respectively.

Values of f may be calculated from the well-known formulas of Bazin or Kutter and Ganguillet. Alternatively, Manning's formula

$$\frac{\delta h - \frac{{v_A}^2 - {v_B}^2}{2g}}{l} = \left(\frac{n}{1{\cdot}486}\right)^2 \frac{v^2}{m^{1{\cdot}34}} \quad . \quad . \quad . \quad . \quad . \quad (2)$$

may be adopted *.

Or, again, in the case of earth channels operating in average working conditions, Barnes' formula

$$\delta h' = \delta h - \frac{{v_A}^2 - {v_B}^2}{2g} = 0{\cdot}000275 \frac{l v^{2{\cdot}016}}{m^{1{\cdot}399}} \quad . \quad . \quad . \quad (3)$$

may be used †.

For this purpose, three classes of earth channel have been considered:

Class A: very regular surface, or revetted with stone, Bazin's $N = 1{\cdot}54$; Kutter's $n = 0{\cdot}0225$.

Class B: ordinary, Bazin's $N = 2{\cdot}35$; Kutter's $n = 0{\cdot}025$.

Class C: exceptionally rough (bed covered with boulders) or weed-grown sides; Bazin's $N = 3{\cdot}17$; Kutter's $n = 0{\cdot}030$.

In each case, Kutter's b has been assumed $= 47{\cdot}2$, corresponding to a gradient of 1 in 2,000; the calculations would not be materially altered by any other reasonable value of b §.

The results of these calculations are tabulated below:

TABLE III.—ESTIMATED LOSS IN FEET IN RIVER MERSEY BETWEEN STATIONS A AND B DUE TO TEXTURAL ROUGHNESS.

Q: cusecs.	Class A.			Class B.				Class C.		
	Bazin.	Kutter.	Manning.	Bazin.	Kutter.	Manning.	Barnes.	Bazin.	Kutter.	Manning.
350 . .	2·78	3·15	3·12	4·29	3·88	3·84	3·47	6·14	5·64	5·55
2,550 . .	6·39	7·71	7·60	9·15	9·41	9·37	8·40	12·4	13·1	13·6
7,000 . .	8·65	10·7	10·2	11·7	12·8	12·5	10·8	15·3	17·4	17·9

Averaging the above figures in each class, and expressing the result as a percentage of the total observed loss of head in the river, gives the figures shown in Table IV (p. 120).

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* A. H. Gibson, "Hydraulics and its Applications." London. (3rd edition), p. 288.
P. à'M. Parker, "The Control of Water." London, 1932, pp. 470 and 1004.

† A. A. Barnes, "Hydraulic Flow Reviewed." London, 1916, p. 46.

§ P. à'M. Parker, "The Control of Water." London, 1932, p. 1004.

TABLE IV.—PERCENTAGE OF LOSS DUE TO TEXTURAL ROUGHNESS.

Q: cusecs.	Class A.	Class B.	Class C.
350	15·1	19·3	28·9
2,550	30·2	37·9	54·3
7,000	33·8	41·4	58·2

It is probable that the general condition of the sides and bed of the river lies intermediate between classes B and C previously defined, and in consequence, the above results would indicate that the percentage loss due to textural roughness is of the order of 25 at dry-weather flow and 45 to 50 for discharges ranging between 2,500 and 7,000 cubic feet per second.

ROUGHNESS-FACTORS.

It is of interest to consider the roughness problem in the light of other experimental investigations. Nikuradse (1931) has measured the loss of head in pipes of various degrees of roughness and has expressed his results in the form of curves connecting λ with Reynolds number. His definition of λ is equivalent to $\lambda = 4f$, where in accordance with the present notation, $\delta h' = flv^2/2gm$.

As a measure of roughness, Nikuradse has adopted the parameter k/r, k denoting the mean height of the projections which constitute the roughening medium, and r the radius of the pipe [1].

His results are of special interest as showing that, for sufficiently high Reynolds numbers, f becomes sensibly constant for a given value of k/r.

Fig. 2 (p. 122) shows Nikuradse's ultimate values of f plotted against m/k, m being the hydraulic mean radius of the pipe.

Superimposed in this diagram are the Author's results from experiments on sand-roughened open channels [2]. In these tests the measured mean height of projection of the sand-grains was 0·03 inch and, as in Nikuradse's work, it was found that f, for a given value of m, settles down to a constant value at sufficiently high Reynolds numbers.

Despite the step from cylindrical pipes to open rectangular channels, and the probability of different shapes of grains and different intensities of roughness in Nikuradse's and the Author's experiments, there is close agreement between the open channels having rough sides and rough bed, and the cylindrical pipes. Comparison is facilitated by the following Table.

[1] Nikuradse's results have been the subject of detailed examination by von Kármán and Prandtl.

[2] Footnote (2), page 115.

TABLE V.

m/k.	Values of friction-coefficient, f.	
	Nikuradse (rough circular pipes).	Author (open channels, rough sides, rough bed).
29·2	0·00895	0·00865
34·5	0·00840	0·00810
41·0	0·00780	0·00745

It appears that the open-channel tests give the same values of f as the pipe-tests if the roughness, k/m, is about 10 per cent. greater than that of the pipes.

Considering next the case of open channels with rough bed and smooth sides, it would be anticipated that if the breadth of stream is great compared with the depth, the resistance will approximate closely to that of an entirely rough section.

The Author's observations[1] confirm this theory, as demonstrated below, f denoting the friction-coefficient for rough sides and bed, and f' the friction-coefficient for smooth sides and rough bed.

TABLE VI.

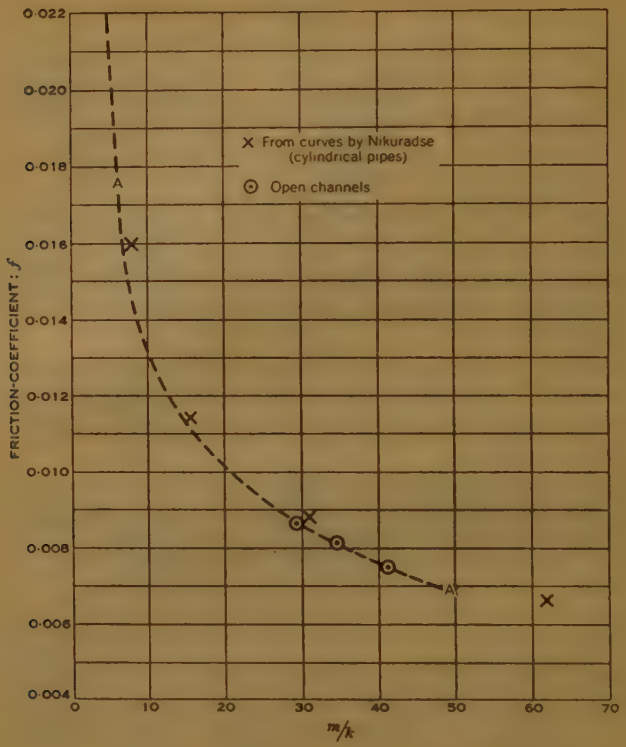
Hydraulic mean radius, m: feet.	Ratio, breadth/depth.	Ratio of $\frac{f}{f'}$
0·0103	0·519	1·16
0·0865	1·03	1·14
0·0730	1·59	1·08

If now, in accordance with the facts concerning the Mersey (pp. 118 *et seq.*) it is assumed that the loss of head due to textural roughness in the river Mersey to be 25·0, 48·5 and 50·0 per cent. of the total drop at discharges of 350, 2,550 and 7,000 cubic feet per second respectively, the corresponding values of f are 0·0151, 0·0111 and 0·00965.

The application of these values to the curve marked AA in *Fig. 2* (p. 122) would make $m/k = 7·0$, 15·2, 22·0 respectively, and such results would lead to an almost constant k of approximately 0·50 foot. According to this reasoning the river is equivalent, as far as its textural roughness is concerned, to a channel of rectangular section roughened with projections about 6 inches high. Having regard to the presence of stones on the bed and to the uneven surface of the earthen sides, such a conclusion is not improbable.

[1] Footnote (2), page 155.

Fig. 2.



General Formula for Total Drop.

Calling h the total observed drop of the water-surface, and

$$h' = h - \frac{v_A^2 - v_B^2}{2g},$$

let

$$\frac{h'}{l} = \mathrm{i}, \text{ and}$$

$$v = Km^{1/x} i^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

It is then found that

$$\left.\begin{aligned} x &= 1{\cdot}63 \\ K &= 28{\cdot}2 \end{aligned}\right\} \text{ for } Q = 350 \text{ cusecs},$$

$$\left.\begin{aligned} x &= 1{\cdot}77 \\ K &= 46{\cdot}2 \end{aligned}\right\} \text{ for } Q = 2{,}550 \text{ cusecs},$$

$$\left.\begin{aligned} x &= 1{\cdot}76 \\ K &= 49{\cdot}2 \end{aligned}\right\} \text{ for } Q = 7{,}000 \text{ cusecs}.$$

Further reference to these values of K and x will be made subsequently in discussing the resistance of the model-river.

The Calibration of the Model.

Immediately after the preliminary moulding of the model, water was supplied at rates corresponding to 2,550 and 7,000 cusecs in nature. The observed total loss of head in the model, over its whole length, was seen to be true to nature within 6 and 12 per cent. respectively. In each case, the model gave too small a drop.

As then tested, the sides and bed of the model were in a rough state, no attempt having then been made to smooth the chisel-marks and the general surface irregularities of the concrete.

The results were encouraging, however, as indicating that, despite the complexity of the hydraulic phenomena involved, it would be readily practicable so to adjust the model as to make it hydraulically similar to the actual river at all points.

First, the artificial projections and indentations of the sides left by the preliminary moulding were carefully smoothed and a thin layer of neat cement was applied. On the whole, this process resulted in a very slight reduction in channel-width, probably by not more than 1 per cent. The channel after this treatment was very considerably smoother in surface than before. Despite this fact, however, the observed total drop over the full length of the model for discharges equivalent to 2,550 and 7,000 cubic feet per second differed from the first observations by less than $\frac{1}{2}$ per cent.

The fear is often expressed that the standard of finish imparted to the cement or concrete surface in a model must have an important bearing on the results obtained. The experiments described above are of interest, therefore, as proving that, in a model of this character, the surface-finish of the concrete itself has, within wide limits, no measurable effect on the total resistance of the channel.

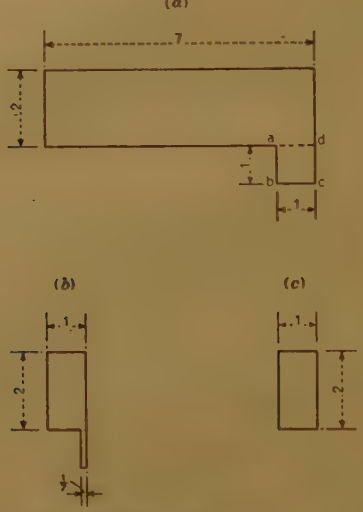
It was clear at this stage, in fact, that minor adjustments of the configuration at the sharp bends would affect the resistance far more than any alteration in surface-roughness. However detailed within practicable limits a survey of such a river may be, it cannot completely describe the degree of severity of the changes of curvature. The criterion for the final moulding of the model at these particular sections must, therefore, be the hydraulic results obtained.

Accordingly, the shape of the bends was next adjusted until a close overall hydraulic agreement between model and nature was reached. Further adjustments of channel-section then made the water-levels at all the gauge-stations still closer to those observed in the natural river for all discharges between 2,550 and 7,000 cusecs. In order to produce similar agreement at a discharge of 350 cusecs, when the average depth

in the model-channel is only of the order of $\frac{1}{2}$ inch, a handful of coarse sand (0·10 by 0·075 by 0·05 inch) was distributed over the bed, by trial and error, until the appropriate water-levels were obtained. The comparison of water-levels measured in the model in its final state with levels at corresponding gauges in the natural river are given in Table VII.

Having thus adjusted the model-channel to give the appropriate water-levels [1], the dimensions of the channel were carefully examined. For this purpose, plasticine templets were pressed to shape to fit the channel-section at each of twenty-four points along its length. From these

Figs. 3.



plasticine templets, the corresponding cross sections were drawn so that areas and hydraulic mean depths could be measured for each section.

It was thus found that, to scale, the mean area of cross section agreed very closely with the natural river, for a given mean depth of water. On the other hand, the mean level of the deepest points in the model-channel was found to be 1·0 feet higher than in nature [2]. For a given mean water-surface level, therefore, the cross-sectional area of the model-channel is less, proportionately, than in the actual river.

A reasonable physical explanation appears to be as follows:

The river-sections in general contain a more or less localized depression represented diagrammatically in *Figs. 3* (*a*).

[1] The engineering purpose of the model being to investigate problems of eliminating the bends by means of straight cuts, it was vitally important to have the correct water-levels at the ends of the cuts.

[2] The width of the water-surface, in the model, averages about 2·18 centimetres with the lowest discharges, and 4·55 centimetres with the highest.

TABLE VII.—COMPARISON BETWEEN LEVELS IN RIVER AND MODEL WITH CORRESPONDING DISCHARGE. (Levels are stated in feet above O.D.)

Section.	Discharge, Q : cusecs. Nature 350.	Model 350.	Nature 2,550.	Model 2,550.	Nature 3,000.	Model 3,000.	Nature 3,420.	Model 3,420.	Nature 4,300.	Model 4,300.	Nature Approx. 7,000 cusecs.		Model 7,000.
Irlam . . .	34·6	34·6	36·1	36·1	36·4	36·2	36·6	36·5	36·9	36·8	38·3	38·3	38·0
4 . . .	—	—	38·0	37·6	—	—	—	—	40·0	39·7	43·0	41·5	42·4
5 . . .	—	—	41·5	41·1	—	—	42·8	42·5	44·5	44·1	45·4	46·6	46·7
Flixton bridge	35·5	35·7	42·9	42·6	43·8	43·6	44·0	44·1	45·4	45·5	48·8	48·8	48·5
8 . . .	37·8	37·7	45·5	45·5	—	—	—	—	49·0	49·2	52·8	52·7	52·9
9 . . .	—	—	47·4	47·3	—	—	—	—	51·0	51·0	55·8	55·0	55·2
10 . . .	42·0	42·0	49·5	49·6	—	—	52·4	51·9	54·5	53·9	57·5	56·3	57·5
11 . . .	—	—	53·5	53·1	—	—	—	—	57·0	56·8	60·0	58·9	59·3
11A . .	47·4	47·3	55·2	54·7	56·6	55·8	56·6	56·7	59·3	58·8	62·4	61·5	62·4
11D . .	—	—	57·1	56·9	—	—	—	—	61·0	60·7	63·6	62·9	63·9
11E . .	—	—	58·5	58·3	—	—	—	—	62·5	62·1	64·9	65·8	65·7
12A . .	—	—	61·3	61·0	61·7	61·9	62·1	62·8	64·5	64·4	68·2	67·2	68·0
155 . . .	56·3	56·3	63·1	62·8	63·8	63·7	63·9	64·5	66·2	66·1	70·0	70·0	70·1
Condition of channel at knife-edge.	Broken†	Broken	Blocked	Blocked	Blocked	Blocked	Broken	Broken	Broken ?	Broken	—	—	—
Date . . .	29/9/36	—	5/12/34	—	2/1/35	—	10/10/35, 29/10/35.	—	16/2/35	—	7/1/32	—	—

* Wrack reading.

† The peninsular near station 11 was cut through by the channel shown in broken lines in *Fig. 1* (p. 117).

If, now, the vertical scale of *Figs. 3* (*a*) is exaggerated 7 : 1 times, the resulting equivalent section is as in *Figs. 3* (*b*). The reproduction of the depression abcd in a small-scale model becomes either uncertain or impracticable, and the section degenerates into that represented by *Figs. 3* (*c*).

The area of *Figs. 3* (*c*), in this hypothetical instance, is 0·935 of the area of *Figs. 3* (*b*), but the wetted perimeter is reduced by 28·5 per cent. In order to compensate for this disproportionate increase of the hydraulic mean depth, it would be necessary to reduce the width of the rectangle, and consequently its area.

In the model now under consideration the comparative areas and hydraulic mean depths are given in Table VIII.

TABLE VIII.

Q : cusecs.	Ratio of actual river to model-river.	
	Mean area.	Mean value of m.
350	104,000	200
2,550	111,000	282
7,000	105,000	308

Now, the nominal scales of the model (1 to 800 horizontally and 1 to 120 vertically) would require cross-sectional areas to be in the ratio of 96,000 to 1. Accordingly, the actual river cross-section is, proportionately, 1·09, 1·16, and 1·10 times that of the model for discharges of 350, 2,550, 7,000 cusecs respectively.

Thus, while maintaining the scalar ratio ($800 \times 120 \times \sqrt{120}$) to 1 for rates of discharge, it has been necessary, due to the complexity of the hydraulic resistance, to adjust the section of the model-channel until it is less than that appropriate to the scale of length and depth.

RESISTANCE OF THE MODEL-CHANNEL.

The effect of the exaggeration of scale vertically, together with the irregularity of the channel cross-sections, is to render the relationship between the model and the natural hydraulic mean depths extremely complex (pp. 123 *et seq.*).

A general inference, however, may be deduced from the data already tabulated. In the model, the hydraulic mean depth does not decrease so rapidly, with reduction in rate of flow, as it does in nature. Correspondingly, it is to be expected that the roughness must increase more rapidly. This effect is borne out by the experiments, as the necessity to roughen the bed with coarse sand has its most marked influence with low depths of stream.

On p. 122 an exponential formula, $v = Km^{1/x}i^{\frac{1}{2}}$, has been given,

covering the whole of the loss of head h' not accounted for by acceleration or retardation of the stream. It is a remarkable fact that this same formula fits the model equally well, as shown below :

TABLE IX.

Discharge equivalent to	Model data.			K.	x.	Calculated h' : feet.
	v : feet per second.	m : feet.	h' : feet.			
350 cusecs.	0·136	0·0175	0·167	28·2*	1·63*	0·166
978 ,, .	0·253	0·0216	0·178	38·4	1·74	0·177
1,670 ,, .	0·327	0·0245	0·190	43·0	1·77	0·189
2,550 ,, .	0·382	0·0268	0·202	46·2*	1·77*	0·202
4,600 ,, .	0·462	0·0304	0·223	49·7	1·77	0·221
7,000 ,, .	0·511	0·0348	0·239	49·2*	1·76*	0·242

* These are the values of K and x previously found to fit the observations in the natural river.

The effect of roughening the bed of the model with coarse sand is demonstrated in Tables X and XI :

TABLE X.—EQUIVALENT FALL IN WATER-SURFACE BETWEEN A AND C (*Fig. 1*, p. 117).

Equivalent Q : cusecs.	h : feet.	
	Smooth bed.	Rough bed.
250	13·2	13·8
300	13·2	13·7
400	13·1	13·6
500	13·1	13·5
1,000	13·0	13·2
2,000	12·7	12·8
3,000	12·5	12·6
4,000	12·4	12·5
5,000	12·3	12·4
6,000	12·3	12·4
7,000	12·2	12·3

ROUGHNESS-FACTORS IN THE MODEL.

If it is assumed, as for the natural river, that 25·0, 48·5, and 50·0 per cent. respectively of the total loss of head is due to textural roughness when the rate of discharge corresponds to 350, 2,550 and 7,000 cubic feet per second, then the equivalent values of f are respectively 0·0513, 0·0230, and 0·0209 for the three different discharges.

TABLE XI.—EQUIVALENT FALL IN WATER-SURFACE BETWEEN A AND B (*Fig. 1*, p. 117).

Equivalent Q cusecs.	h : feet.	
	Smooth bed.	Rough bed
250	18·2	19·6
300	18·4	19·8
400	18·8	20·0
500	19·0	20·2
1,000	20·4	21·2
2,000	22·4	23·0
3,000	24·1	24·6
4,000	25·3	25·9
5,000	26·5	27·1
6,000	27·6	28·1
7,000	28·6	29·0

It is not unreasonable to extrapolate the curve AA in *Fig. 2* (p. 122) to embrace the two lower values of f (0·0209 and 0·0230). If this extrapolation is accepted, values of m/k are obtained equal to about 5·0 and 4·0 respectively.

These estimates of m/k would make k = approximately 0·08 inch, and the model-channel becomes equivalent, as far as its surface-roughness is concerned, to a rectangular section with projections of the order of 0·08 inch in height. In view of the coarse sand (dimensions 0·10 by 0·075 by 0·05 inch) used to roughen the bed, and the actual departure from a purely rectangular cross section, such a figure is quite appropriate.

EFFECT OF VISCOSITY.

For water at a temperature of 15° C., the Reynolds number (vm/ν) corresponding to various rates of flow in model and in nature is as follows:

TABLE XII.

Discharge in nature: cusecs.	Reynolds number.	
	Nature.	Model.
350	$0{\cdot}388 \times 10^6$	193
2,550	$2{\cdot}20 \times 10^6$	827
7,000	$4{\cdot}41 \times 10^6$	1,440

The relatively low Reynolds numbers in the model-experiment

make it essential to determine whether any scale-effect is present due to changes in viscosity.

Experiments have accordingly been made with hot and cold water. In each case the water-temperature was measured, by calibrated thermometer, at five points along the channel. Measurements of water-surface level at a number of gauge-points were made, but for the purpose of discussion, attention is confined to the three positions marked A, B, and C in *Fig. 1* (p. 117).

Measuring along the channel centre-line, AB = 1,510 centimetres = 49·5 feet, and AC = 719 centimetres = 23·6 feet. The corresponding distances in the natural river are, therefore, 39,600 and 18,900 feet respectively.

The water-level at station B was sensibly the same (for a given discharge) with cold and hot water, as also was the drop in level from A to C. The fall between A and B, however, was very slightly reduced when hot water was used, as will be seen from Table XIII. In preparing this Table, the experimental results have been set out against discharge, and values for convenient rates of flow read from the curves. Values are given in units corresponding to the full-size river. A discharge of 1,000 cusecs represents, therefore, 26·8 cubic centimetres per second in the model; a drop in water-level, h, of 20·0 feet, is the equivalent of 5·06 centimetres in the model.

TABLE XIII.—RESULTS OBTAINED IN MODEL WHEN OPERATED WITH HOT AND COLD WATER.

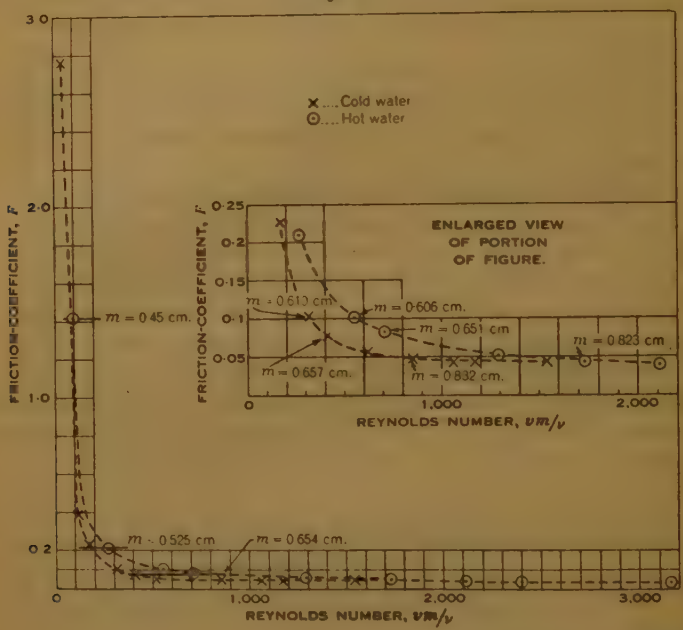
Discharge, Q: cusecs.	h between A and B: feet.		Average viscosity, ν.	
	Cold.	Hot.	Cold.	Hot.
50	19·2	19·2	0·0122	0·0076
150	19·6	19·4	0·0126	0·0076
250	19·6	19·6	0·0125	0·0076
300	19·8	19·7	0·0125	0·0076
400	20·0	19·9	0·0125	0·0076
500	20·2	20·1	0·0124	0·0073
1,000	21·2	21·0	0·0122	0·0070
2,000	23·0	22·7	0·0120	0·0059
3,000	24·6	24·3	0·0117	0·0059
4,000	25·9	25·6	0·0110	0·0055
5,000	27·1	26·9	0·0112	0·0056
6,000	28·1	27·9	0·0110	0·0054
7,000	29·0	28·8	0·0108	0·0052

Inspection of the data given above shows that, despite the great

difference in viscosity (from 60 to 100 per cent.) the observed resistance is not affected by more than 1 or 2 per cent.

If values of F, the friction-coefficient in $h' = Flv^2/2gm$, are plotted against vm/ν, *Fig. 4* is obtained, showing two distinct curves for hot and cold water. The interpretation to be placed on this diagram, however, appears to be as follows :

Fig. 4.



The results should not be connected by the approximately hyperbolic curves shown in broken lines, but by a family of full lines corresponding to different values of m. Such a process is analogous to that in turbulent flow along pipes or straight open channels of different degrees of roughness.

A further physical explanation is offered : accepting the formula $v = Km^{1/x}i^{\frac{1}{2}}$ as already mentioned, and assuming K to be proportional to $g^{\alpha}\nu^{\beta}$, dimensional analysis indicates that

$$\text{if } x = 1{\cdot}63, \text{ then } \alpha = 0{\cdot}537 \text{ and } \beta = -0{\cdot}076,$$
$$\text{or if } x = 1{\cdot}77, \text{ then } \alpha = 0{\cdot}521, \text{ and } \beta = -0{\cdot}034.$$

Corresponding to $\beta = -0{\cdot}076$, a 100-per-cent. rise in viscosity would entail an increase of surface-gradient of some 15 per cent., but would involve increasing the general depth of the stream. Such an increase of

depth would reduce the velocity and increase m, both effects being such as to favour a reduction in gradient.

Since, for a given downstream level near Irlam weir, the mean m in the model is a function of i, say $\phi(i)$, and the mean velocity, v, is also $\psi(i)$, the gradient is re-arranged by change of viscosity until the equation

$$\psi(i) = k\{\phi(i)\}^{1/x}\nu^{\beta}i^{\frac{1}{2}}$$

is again satisfied.

Summary.

(*a*) Analysis of observations in the stretch of the river Mersey shown in *Fig. 1* (p. 117) indicates that although the river-section is equivalent to a rectangular channel roughened with projections about 6 inches high, its textural roughness accounts only for about 25 per cent. of its total resistance at a flow of 350 cusecs. Even with a discharge as high as 7,000 cusecs, the bends are responsible for as much as one-half of the total loss of head.

(*b*) The total drop in water-surface between points A and B (*Fig. 1*) may be represented by a formula $v = Km^{1/x}i^{\frac{1}{2}}$, where the same values of K and x are applicable to the river itself and to its scale model, at corresponding rates of flow.

(*c*) The method of adjusting and calibrating the model for a wide range of discharge is described in the Paper.

(*d*) Experiments conducted in the model with hot and cold water (viscosity changing by from 60 to 100 per cent.) show almost identical losses of head, despite the Reynolds numbers involved being as low as $vm/\nu = 36$.

Acknowledgements.

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The Paper is accompanied by five sheets of drawings, from some of which the Figures in the text have been prepared, and by the following Appendix.

APPENDIX

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Paper No. 5204.

"Turbulent Flow in Pipes, with particular reference to the Transition Region between the Smooth and Rough Pipe Laws."

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(*Ordered by the Council to be published with written discussion.*)[1]

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INTRODUCTION.

The problem of flow in pipes is one which has until recently defied theoretical analysis, owing to its complexity and the absence of a rational basis for its solution. An outstanding contribution to the knowledge of the subject was made more than half a century ago by Professor Osborne Reynolds, who succeeded in finding a unifying principle which considerably simplified the analysis of his experimental results. His discovery that the change from streamline to turbulent flow depended on the value of $\frac{\rho U d}{\mu}$ led later workers to the corollary that the coefficient λ in the well-known pipe-formula $h = \frac{\lambda l U^2}{2gd}$ is a function of the parameter $\frac{\rho U d}{\mu}$, which was named after him the Reynolds number.

His discovery of this criterion led to the formulation of a more general

[1] Correspondence on this Paper can be accepted until the 15th May, 1939, and will be published in the Institution Journal for October 1939.—SEC. INST. C.E.

"Principle of Dynamical Similarity," which determines the conditions for mechanical similarity in the motions in or around geometrically similar bodies.

Considerations of dynamical similarity may be replaced by dimensional reasoning which leads to a grouping of the quantities involved in the problem into a number of non-dimensional parameters; this enables experimental results to be plotted in a systematic manner. Such considerations, however, have definite limitations since the functional relationship between these groups and their relative importance cannot be determined by dimensional reasoning.

It has been suggested as a result of experiments on lead and other smooth pipes that the resistance-coefficient λ and the Reynolds number R could be expressed satisfactorily by an exponential formula of the type

$$\lambda = AR^n$$

By re-arrangement of this equation into the form

$$i = A_1 \frac{U^x}{d^y}$$

it is easy to show that for smooth pipes the sum of the indices of U and d must be 3 for all pipe-sizes and velocities. This equation is widely known and the argument is frequently put forward that the sum of the indices must equal 3 in any exponential formula designed to fit experimental results on a few pipes over a limited range of velocities of flow. Although this relation between the indices is true for smooth pipes, the value of n itself so depends on the Reynolds number that a single value of n will only give approximately correct results over a limited range of Reynolds numbers. When the roughness-factor is introduced the relation no longer holds: indeed, it will be shown in a later paragraph that, whatever the roughness, this sum always exceeds 3. F. C. Scobey attempts to justify by dimensional reasoning [1] his formula for riveted steel pipes in which the sum of the indices is 3, but his omission, from the argument, of the roughness-factor, which is particularly important in the case of riveted pipes, seriously affects the value of the formula.

In brief, it may be stated that the principle of dynamical similarity determines the non-dimensional parameters governing fluid motion, but fails to determine the functional relationship between them. This has led to a reconsideration of the fundamentals of the problem, and the recent success of L. Prandtl and von Karman in Germany, and of G. I. Taylor in Great Britain, in expressing in mathematical form the mechanism of turbulence,

[1] "Riveted Steel and Analogous Pipes." Bulletin No. 150, Department of Agriculture, U.S.A., 1930.

linked with the experimental investigations of Nikuradse, have now provided a fundamental basis for the analysis of the problem.

They developed a formula of the type

$$\frac{1}{\sqrt{\lambda}} = 2 \log 0{\cdot}113 \frac{d}{y_1} \quad \ldots \ldots \ldots \quad (1)$$

and showed that the lower limit of integration y_1 is a function of the wall-particle size k in the case of rough pipes in which the flow obeys the " square " resistance-law, and is dependent on the density ρ, the viscosity μ, and the shear stress at the wall τ, in the case of smooth pipes.

Substituting appropriate values of y_1 in (1) the following resistance laws are obtained for

(*a*) flow in hydraulically smooth pipes:

$$\frac{1}{\sqrt{\lambda}} = 2 \log \frac{R\sqrt{\lambda}}{2{\cdot}51} \quad \ldots \ldots \ldots \quad (2)$$

(*b*) flow in rough pipes:

$$\frac{1}{\sqrt{\lambda}} = 2 \log 3{\cdot}7 \frac{d}{k} \quad \ldots \ldots \ldots \quad (3)$$

The experimental results of Nikuradse show complete agreement with the above laws provided certain limiting conditions are satisfied. The experiments show that the rough-pipe law is true for values of $\frac{\rho V_* k}{\mu}$† exceeding 60, whilst for values less than 3 even rough pipes obey the smooth-pipe law as the excrescences then cease to contribute to the resistance. Between these values there is a transition from one law to the other.

The smooth, rough, and transition laws for Nikuradse's sand roughness in which the grains are of uniform size and closely packed together, are shown in *Fig. 1* (p. 136) together with the transition curve for a pipe having a roughness composed of isolated particles, the experiments on which are

---

† $V = \sqrt{\frac{\tau}{\rho}}$ and is called the " shear force " velocity, since it has the dimensions of a velocity.

The roughness Reynolds number $\frac{\rho V_* k}{\mu}$ may be expanded into

$$\frac{\rho V_* k}{\mu} = \sqrt{\frac{\lambda}{8}} \cdot \frac{k}{d} \cdot \frac{Ud}{\nu}$$

It will be seen that it is the product of three dimensionless numbers, the resistance-coefficient, the relative roughness, and the Reynolds number.

described in detail elsewhere.[1] It is apparent that with non-uniform roughness the transition zone extends over a range about 10 times as long as that for uniform sand roughness, and in the case of new commercial pipes in which the roughness is non-uniform the whole working range lies within the transition zone. The mean transition curves for galvanized-, cast-, and

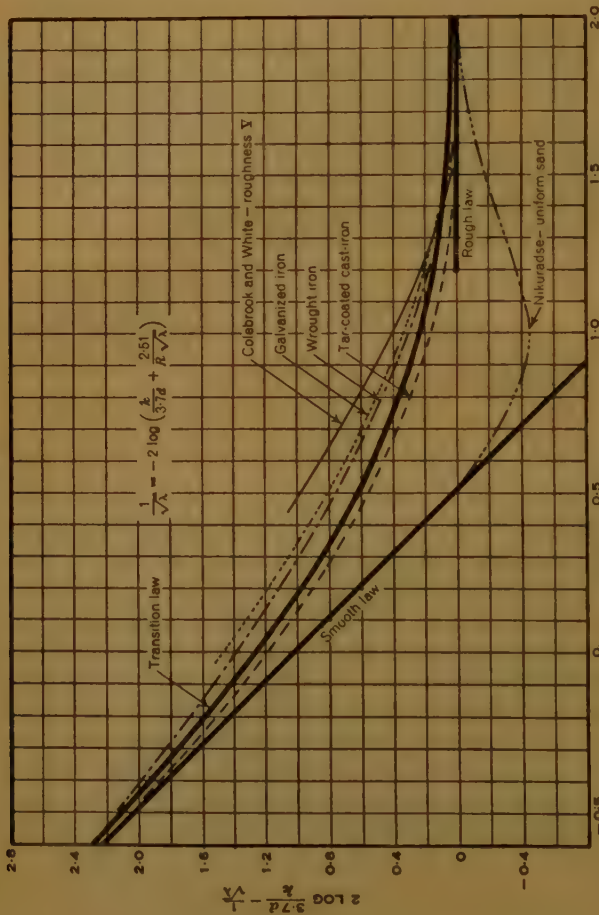


Fig. 1. THEORETICAL AND EXPERIMENTALLY DETERMINED "TRANSITION" LAWS FOR VARIOUS ROUGH SURFACES.

wrought-iron pipes, which were determined by an analysis of most of the available reliable data and described later in the Paper, are shown in *Fig. 1* for comparison with that for the roughness *V*.

[1] C. F. Colebrook and C. M. White, "Experiments with Fluid-Friction in Roughened Pipes." Proc. Roy. Soc. (A), vol. 161 (1937), pp. 367, 381. (See Roughness "V" in this Paper.)

Any attempt to express mathematically the transition-function for uniform sand-roughness is rendered difficult owing to the fact that the turbulent motion in the wake behind the grains is complicated by mutual interference, and the resistance mechanism is made up of viscous and mechanical forces which are difficult to separate.

In the case of non-uniform roughness, however, the large isolated grains have a shielding effect on the smaller grains which considerably reduces their effectiveness so far as total resistance is concerned, so that the area of the pipe between the large excrescences may be regarded as behaving as a smooth surface with a coefficient of resistance dependent on the Reynolds number $\frac{\rho V_* d}{\mu}$. Since the local Reynolds number on the large grains is comparatively large even at fairly low mean velocities, the local grain co-efficient is practically constant over the entire transition range. In effect, $\frac{y_1}{d}$ in (1) is a function of $\frac{k}{d}$, the relative roughness, and $\frac{\mu}{\rho V_* d}$, and has definite limiting values corresponding at the one extreme to fully-rough-law flow-conditions in which viscous resistance is negligible, and at the other extreme to smooth-pipe conditions when the resistance mechanism is entirely molecular. The exact form of the function will depend on the distribution of the roughness-elements and is mathematically indeterminate, but it will be shown in the present Paper that it is possible to obtain a particular transition law which is similar to those obtained experimentally for commercial pipes by simply adding together [1] the lower limits of integration y_1 which satisfy the rough- and smooth-pipe laws. The following general formula is then obtained :

$$\frac{1}{\sqrt{\lambda}} = -2 \log \left(\frac{k}{3{\cdot}7d} + \frac{2{\cdot}51}{R\sqrt{\lambda}} \right) \quad . \quad . \quad . \quad . \quad . \quad (4)$$

which is in exact agreement with theory at extreme values of $\frac{\rho V_* k}{\mu}$ and gives results in the transition-zone which approximate very closely to the experimental values. It will be seen in *Fig. 1* that this transition-curve merges asymtotically into the smooth- and rough-law curves.

Theory of Turbulent Flow in Pipes.

In turbulent motion it has been observed that the velocity-distribution

[1] This treatment of the lower limits of integration was suggested by Dr. C. M. White, and the Author desires to place on record his indebtedness to Dr. White for his collaboration in the development of formula (4).

may be expressed by the relation

$$\frac{du}{dy} = \sqrt{\frac{\tau}{\rho}} \frac{2{\cdot}5}{y} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

where u denotes the velocity at a distance y from the wall of the pipe, τ, the shear stress at the wall, and ρ, the density of the fluid.

On integration the equation (5) becomes

$$u = 5{\cdot}76\sqrt{\frac{\tau}{\rho}} \log \frac{y}{y_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Since $u = 0$ when $y = y_1$ the effective hydraulic wall may be regarded as being displaced inwards from the actual wall by an amount y_1. The hydraulic wall then represents a plane where the disturbances are theoretically as great as the actual ones at the wall.

The mean velocity U is numerically equal to the local velocity at $y = 0{\cdot}113d$*, and substituting this value of y in (6), the equation becomes

$$U = 5{\cdot}76\sqrt{\frac{\tau}{\rho}} \log 0{\cdot}113 \frac{d}{y_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

Re-arranging (7) so as to introduce the resistance-coefficient into the equation,

$$\frac{1}{\sqrt{\lambda}} = 2 \log \frac{0{\cdot}113d}{y_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

Equation (8) may be regarded as a general formula applicable to all types of turbulent flow in pipes. The shift of the effective hydraulic wall y_1 has, however, to be determined in order completely to determine the resistance-law. Since y_1 depends on the conditions at the wall it must clearly be a function of (*a*) the roughness of the wall k, (*b*) the shear-stress τ, and (*c*) the kinematic viscosity of the fluid, $\nu = \frac{\mu}{\rho}$.

It has been observed experimentally that providing $\frac{\rho V_* k}{\mu}$ exceeds about 60 the resistance is proportional to the square of the velocity (that is, the resistance-coefficient is independent of the viscosity of the fluid), and in this case dimensional reasoning shows that the shift y_1 can only be proportioned to k. Nikuradse, experimenting with pipes artificially roughened internally by a uniform layer of sand fixed to the walls, deter-

* For the proof of this expression, see "The Reduction of Carrying Capacity of Pipes with Age," by C. F. Colebrook and C. M. White. Journal Inst. C.E., vol. 7 (1937–38), p. 99. (November 1937.)

mined a value of

$$y_1 = \frac{k}{33},$$

where k denotes the diameter of the sand grains. Inserting this value of y_1 in (8), the resistance-law for rough pipes becomes

$$\frac{1}{\sqrt{\lambda}} = 2 \log 3{\cdot}7 \frac{d}{k} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

In the case of smooth pipes (or rough pipes when $\frac{\rho V_* k_{\max.}}{\mu}$ is less than 3 when the roughness particles cease to shed eddies and contribute to the resistance), the resistance is due entirely to molecular or viscous mixing, and y_1 must, by dimensional reasoning, be proportional to $\frac{\mu}{\rho V_*}$, which is the only combination of τ, ρ, and μ which has the same unit as a length.

Other experiments by Nikuradse show that for smooth pipes

$$y_1 = \frac{1}{10} \frac{\mu}{\rho V_*}$$

which on insertion in (8) leads to the resistance-law for smooth pipes

$$\frac{1}{\sqrt{\lambda}} = 2 \log \frac{R\sqrt{\lambda}}{2{\cdot}51} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

When $\frac{\rho V_* k_{\max.}}{\mu}$ exceeds 3, however, the resistance increases over that of a smooth pipe due to the shedding of eddies by the roughness-protuberances.

A New Theoretical Formula for Flow in the Transition Region.

The value of y_1 may be regarded as having two extremes which satisfy the smooth-law and fully rough-law conditions respectively, whilst in the transition range y_1 exceeds both of these extreme values due to a combination of mechanical and viscous mixing at the walls.

Thus,

$$y_1 = \phi\left(k, \frac{\mu}{\rho V_*}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

Putting (9) into non-dimensional form, the equation becomes

$$\frac{y_1}{d} = \phi\left(\frac{k}{d}, \frac{\mu}{\rho V_* d}\right) . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Analytically, equation (10) must take the form

$$\frac{y_1}{d} = \alpha\frac{k}{d} + \beta\frac{\mu}{\rho V_* d} \quad \ldots\ldots\ldots \quad (11)$$

where α and β are numerical constants to be found by experiment.

For pipes having non-uniform roughness k may be regarded as being the roughness of a sanded surface giving the same resistance-coefficient as the non-uniformly roughened surface.

Nikuradse's values for α and β are $\frac{1}{33}$ and $\frac{1}{10}$ respectively, and substituting these numerical values in (11) and inserting the resulting value of y_1 in (8) the resistance-law becomes

$$\frac{1}{\sqrt{\lambda}} = 2\log\frac{0{\cdot}113d}{\frac{k}{33} + \frac{1}{10}\cdot\frac{\mu}{\rho V_*}}$$

$$= -2\log\frac{1}{0{\cdot}113}\left(\frac{k}{33d} + \frac{1}{10}\cdot\frac{\mu}{\rho V_* d}\right)$$

which may be rewritten as

$$\frac{1}{\sqrt{\lambda}} = -2\log\left(\frac{k}{3{\cdot}7d} + \frac{2{\cdot}51}{R\sqrt{\lambda}}\right) \quad \ldots\ldots \quad (12)$$

In order to represent (12) graphically it is convenient to separate the independent variable $\frac{\rho V_* k}{\mu}$ from the remainder. Thus,

$$-\frac{1}{\sqrt{\lambda}} + 2\log\frac{3{\cdot}7d}{k} = 2\log\left(\frac{3{\cdot}28}{\frac{\rho V_* k}{\mu}} + 1\right) \quad \ldots\ldots \quad (13)$$

This function is shown as a heavy line in *Fig. 1.* (p. 136). It will be noticed that the theory indicates a slight increase in resistance over that for purely rough-law flow at $\frac{\rho V_* k}{\mu} = 60$, but this discrepancy against experiment is very small and diminishes with increasing values of $\frac{\rho V_* k}{\mu}$. The curve approaches the smooth- and rough-laws asymtotically in accordance with experimental observation.

The formula for flow in smooth pipes

$$\frac{1}{\sqrt{\lambda}} = -2\log\frac{2{\cdot}51}{R\sqrt{\lambda}} \quad \ldots\ldots\ldots \quad (2)$$

is rather inconvenient for practical use since the resistance-coefficient appears on both sides of the equation. This difficulty is overcome by using the formula

$$\frac{1}{\sqrt{\lambda}} = 1{\cdot}8 \log \frac{R}{7} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

which is a mathematical approximation to the exact formula (2) but gives numerical results within $\pm \frac{1}{2}$ per cent. over a range of Reynolds numbers of from 5,000 to 100,000,000.

Relation between Prandtl–von-Karman and Exponential Formulas.

It is of interest to compare the results obtained by the modern rational method of analysis of the problem of fluid-flow with the earlier empirical formulas of the exponential type. The Prandtl–von-Karman rough pipe-law $\frac{1}{\sqrt{\lambda}} = 2 \log 3{\cdot}7 \frac{d}{k}$ may be converted to the exponential type

$$\frac{1}{\sqrt{\lambda}} = A\left(\frac{d}{k}\right)^n \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

by taking logarithms and differentiating. Thus

$$n = \frac{d\left(\log \frac{1}{\sqrt{\lambda}}\right)}{d\left(\log \frac{d}{k}\right)} = 0{\cdot}87\sqrt{\lambda} \quad . \quad . \quad . \quad . \quad (16)$$

Hence

$$\frac{1}{\sqrt{\lambda}} = A\left(\frac{d}{k}\right)^{0{\cdot}87\sqrt{\lambda}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

Formula (17) may be extended into the usual form

$$U = \left(\frac{A\sqrt{2g}}{k^{0{\cdot}87\sqrt{\lambda}}}\right) d^{0{\cdot}5+0{\cdot}87\sqrt{\lambda}}\, i^{0{\cdot}5}$$

or

$$i = \left(\frac{k^{1{\cdot}74\sqrt{\lambda}}}{A^2 \,.\, 2g}\right) \frac{U^2}{d^{(1+1{\cdot}74\sqrt{\lambda})}}$$

It is clear that the exponent, n, is itself a function of the resistance-coefficient, so that a single value will only give approximately correct results over a limited range of d/k values. In order to illustrate the argument, suppose it is necessary to develop exponential formulas to cover a

range of $d/k = 10$ to 40,000 so as to give results to within $\pm 2\frac{1}{2}$ per cent of the correct value. It will be found by plotting $\log \frac{1}{\sqrt{\lambda}}$ against $\log \frac{d}{k}$ that it is necessary to divide up the range into two components of $d/k = 10$ to 200 and $d/k = 200$ to 40,000. The values of A and n then become

$$d/k = 10 \text{ to } 200,\ A = 2{\cdot}03, \text{ and } n = 0{\cdot}20$$
$$d/k = 200 \text{ to } 40{,}000,\ A = 3{\cdot}25, \text{ and } n = 0{\cdot}111$$

It is to be noted that the sum of the indices of U and d always exceeds 3 in the case of rough pipes by $1{\cdot}74\sqrt{\lambda}$.

An exponential formula of the type

$$\frac{1}{\sqrt{\lambda}} = AR^n \quad \ldots\ldots\ldots \quad (18)$$

may be developed from the Prandtl–von-Karman smooth-pipe law $\frac{1}{\sqrt{\lambda}} = 2 \log \frac{R\sqrt{\lambda}}{2{\cdot}51}$ by taking logarithms and differentiating. The exponent n is given by $\frac{d\left(\log \frac{1}{\sqrt{\lambda}}\right)}{d(\log R)}$, which becomes

$$n = \frac{1}{1 + \frac{1{\cdot}15}{\sqrt{\lambda}}} \quad \ldots\ldots\ldots \quad (19)$$

Thus

$$\frac{1}{\sqrt{\lambda}} = AR^{\frac{1}{1+\frac{1{\cdot}15}{\sqrt{\lambda}}}} \quad \ldots\ldots\ldots \quad (20)$$

Equation (20) on extension becomes

$$U = \left(\frac{A\sqrt{2g}}{\nu^n}\right)^{\frac{1}{1-n}} i^{\frac{1}{2-2n}} d^{\frac{1+2n}{2-2n}}$$

or

$$i = \left(\frac{\nu^{2n}}{A^2 \cdot 2g}\right) \cdot \frac{U^{2-2n}}{d^{1+2n}}$$

where n is given by (19). Here again it is seen that the exponent n is a function of the resistance-coefficient, but in this case the sum of the indices of U and d equals 3 as predicted by dimensional analysis.

This development of the relationship between rational and exponential formulas shows quite clearly that single values of the exponent, n, can only give approximately correct results over a limited range of pipe-sizes, and

velocities and exponential formulas are not, therefore, capable of universal application.

Analysis of Experimental Data on Smooth Pipes.

A number of commerical pipes may be regarded as hydraulically smooth, at least for all ordinary velocities of flow. Among these may be included good commercial drawn-brass pipes, lead, glass, or tin pipes, centrifugally-spun lined (with bitumen or concrete) cast-iron pipes, and concrete-lined pipes which have been deposited against oiled steel forms and carefully rubbed down to remove any imperfections.

The results of an analysis of much of the available experimental data are shown in *Figs. 2* (p. 144), and are seen to be in close agreement with the Prandtl–von-Karman smooth-pipe law.

The data include the experimental results on only one brass pipe of 0·5 inch diameter, obtained at the National Physical Laboratory by Stanton and Pannell in 1915, although the results for a large number of brass pipes of other diameters tested by them also agree very closely with theory. The results on sixteen spun concrete-lined pipes and on six spun bitumastic-lined pipes ranging in size from 4 inches to 60 inches in diameter are included. Of these, the laboratory tests by M. L. Enger on 4-inch, 6-inch and 8-inch pipes were probably subject to the least experimental error, and the results exhibit only slight scatter from the theoretical law. In analyzing the data obtained by B. W. Bryan on the Stour Supply, Danbury to Herongate main, which included one hundred and ninety-two lobster-back bends of radius $3\frac{1}{2}d$, and having a total change in direction of 2,987 degrees, an allowance of $20\frac{U^2}{2g}$ was made in the calculations for bends.

The results on the 216-inch diameter Ontario tunnel, the biggest of its kind in the world, are especially interesting, as particular care was taken in its construction and the range of test-velocities was large. The concrete used in the construction of the tunnel was deposited against oiled steel forms which resulted in a smooth and even surface. All defects were then removed and the surface rubbed down with carborundum brick. In analyzing the test-data [1], it was found that an arithmetical error had been made in calculating the resistance-coefficients in $h = \frac{\lambda l U^2}{2gd}$.

The correct values, which are considerably lower than those given by Scobey, are shown in Table I (p. 145) together with the test-results from which they were computed.

Despite an appreciable experimental scatter the test-results are in very satisfactory agreement with theory.

[1] F. C. Scobey, "Concrete Pipes." Department of Agriculture, U.S.A., Bulletin No. 852.

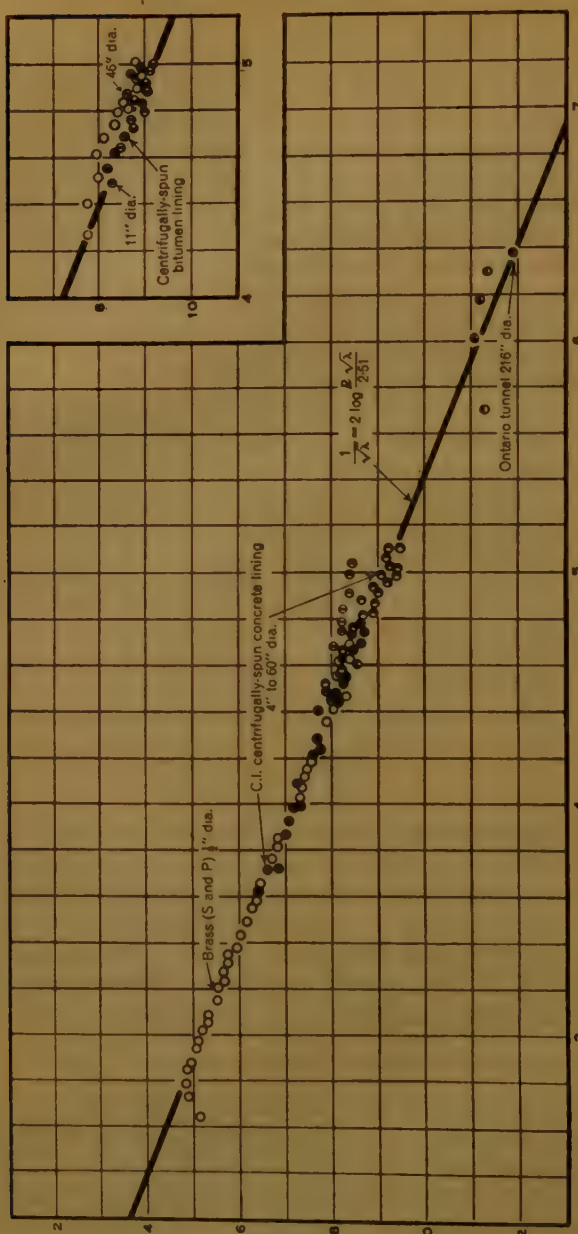


Fig. 2.

COMPARISON BETWEEN THEORY AND EXPERIMENT ON SMOOTH COMMERCIAL PIPES.

TABLE I.

D: inches.	Q: cusecs.	U: feet per second.	Loss in head per 1,000 feet: feet,	Reynolds number.	Coefficient of friction, λ.
216	1,018	4	0·108	5,550,000	0·00782
	2,036	8	0·448	11,100,000	0·00812
	3,045	12	0·990	16,650,000	0·00798
	4,063	16	1·701	22,200,000	0·00773
	5,091	20	2,397	27,700,000	0·00697

GALVANIZED, CAST- AND WROUGHT-IRON PIPES.

In analyzing the data on the various types of iron pipes it was necessary to determine both the mean hydraulic-roughness, k, and the mean transition law for each class. The problem is complicated by the fact that in practice there are variations of roughness due to non-uniformity in the method of manufacture so that in each class there is considerable variation both in the size and type of roughness. It was necessary, therefore, to determine the transition law and roughness k for each individual pipe—a task which is rendered difficult by the fact that with one or two exceptions the experimental results do not cover a wide enough range and rarely reach square-law. However, the experiments on pipe V † indicate fairly rapid transition to the square-law at the higher values of $\frac{\rho V_* k}{\mu}$, and thus with many of the test-results it is possible to extend them with very little error so as to reach square-law and enable the determination of the k values, and thus locate the test-results in the transition-range.

The experimental results for each class of pipe are plotted in *Figs. 3*, *5*, and *7* (pp. 146 *et seq.*) with $\frac{1}{\sqrt{\lambda}}$ as ordinate against $\log R\sqrt{\lambda}$ as abscissa. This arrangement gives a sloping straight line for the smooth-law flow and a series of parallel horizontal lines in the square-law region which extends to the right of the dotted line representing the lower limit of rough-law flow.

The results may be brought to a single line in the rough-law region by plotting $2 \log \frac{3{\cdot}7d}{k} - \frac{1}{\sqrt{\lambda}}$ as a function of $\log \frac{\rho V_* k}{\mu}$. This has been carried out in *Figs. 4*, *6*, and *8* (pp. 147 *et seq.*), and a mean transition curve drawn in for each class of pipe. The k-values determined for all pipes are shown in *Figs. 9* (p. 152), and using the mean k-value for each class together with the corresponding mean transition curve, a number of transition curves have been drawn in *Figs. 3*, *5*, and *7* for direct comparison with the

† Footnote (1), p. 136.

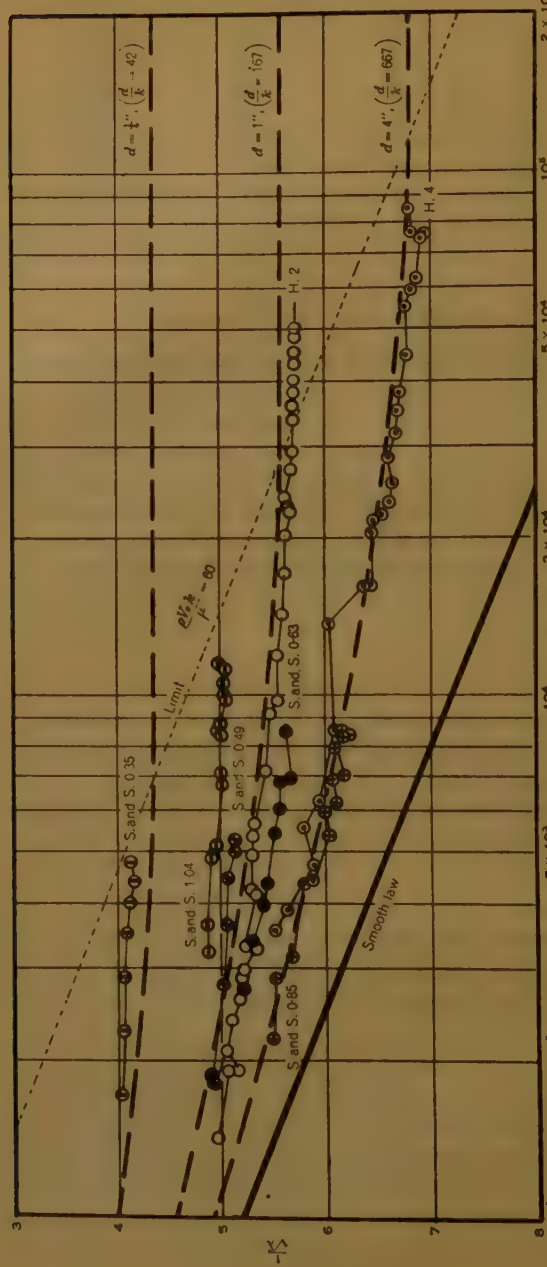


Fig. 3.

EXPERIMENTAL DATA ON GALVANIZED-IRON PIPES.

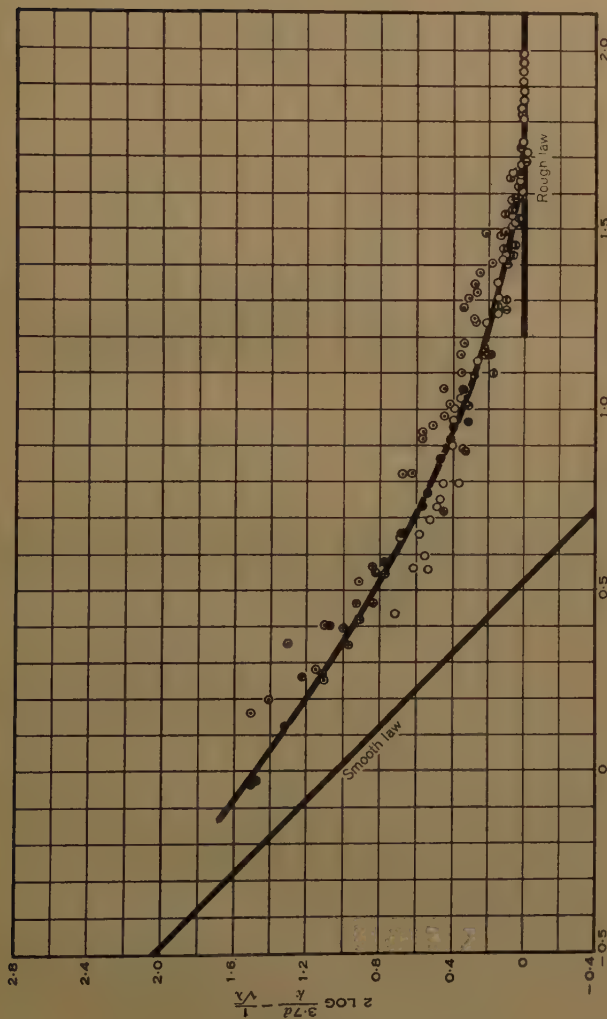
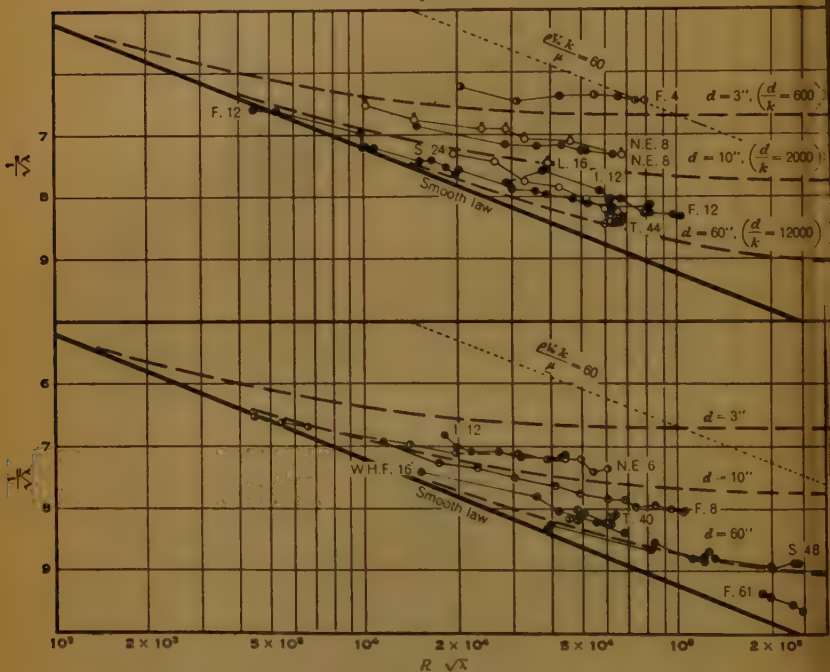


Fig. 4.

DEVIATIONS FROM ROUGH-LAW FOR GALVANIZED-IRON PIPES.

test-results. It is seen that although some of the pipes do not agree ver closely with the mean curves, some having too rapid transition and othe too slow, there appears to be sufficient positive evidence to justify th adoption of the given mean transition laws together with the mean k-value It is to be expected that these will enable the prediction of resistance coefficients in pipes of sizes other than those tested and at velocities beyon the normal range with less uncertainty than with any existing empiric

Fig. 5.



EXPERIMENTAL DATA ON TAR-COATED CAST-IRON PIPES.

formula. With regard to the experimental data itself, space prohibits detailed description of all the data available, so remarks will be confined t a few observations with regard to the most accurate data.

The experiments made by F. Heywood on new galvanized-iron pipe of 2 inches and 4 inches diameter were carefully conducted and are mos valuable, as the range of velocities was very wide, being from 0·5 to 21 fee per second. Referring to *Fig. 3* it will be noticed that the resistance coefficient for the 2-inch pipe becomes constant at high velocities, thu enabling the determination of k and the major portion of the transitio curve. At the lower values of $\frac{\rho V_* k}{\mu}$ the 2-inch and 4-inch pipes diverg

in opposite directions from the mean curves, and the 2-inch pipe is somewhat rougher than the 4-inch.

The remaining data on galvanized pipes was obtained by Saph and Schoder, but in the determination of the mean value of k for this class the

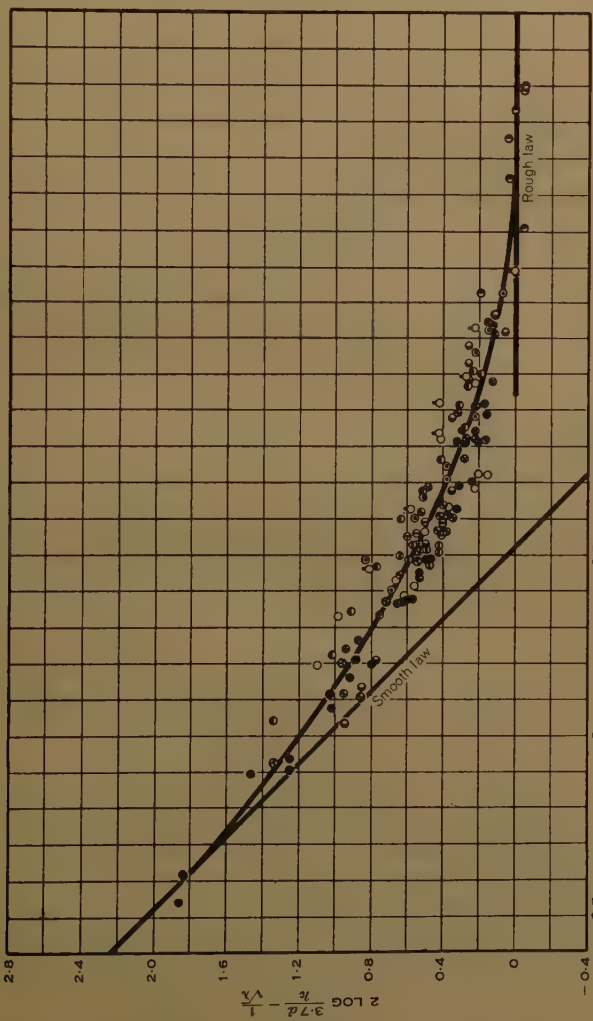


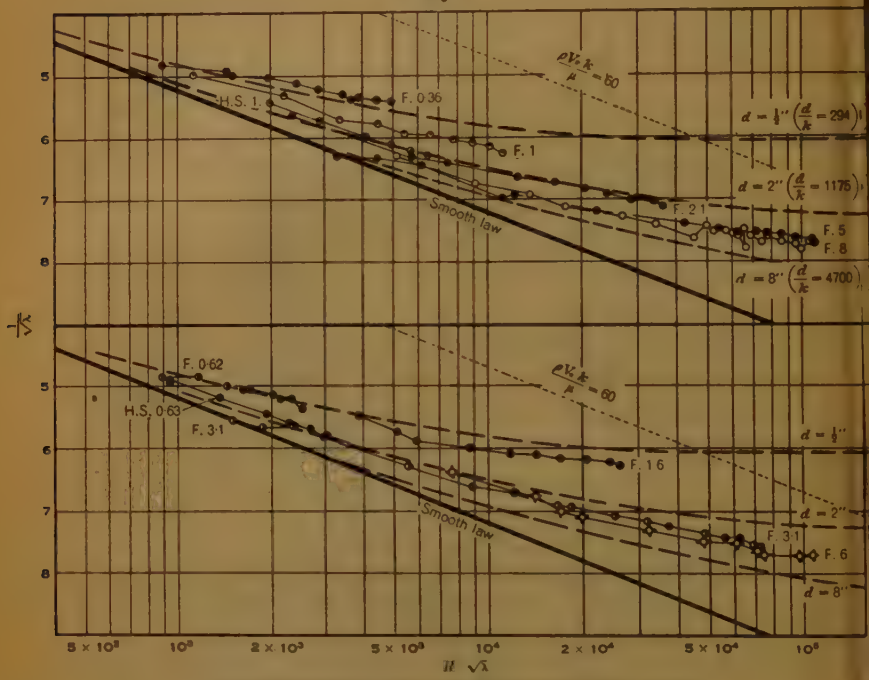
Fig. 6.

DEVIATIONS FROM ROUGH-LAW FOR TAR-COATED CAST-IRON PIPES.

Author has neglected pipe XVIII (0·85 inch in diameter) as the experimenters make the following statement concerning this pipe—"Pipe XVIII (0·85 inch in diameter) seems to be an exceptional pipe, but it has to be remembered that a slight silt-like deposit had occurred on the inner walls which was entirely sufficient to relieve the roughness."

Very reliable data, used in the present analysis on tar-coated cast-iron pipes, was obtained by J. Freeman and H. Mills at Lawrence, Massachusetts on pipes of 4 inches, 8 inches, and 12 inches diameter, and another carefully made experiment was that on a 6-inch pipe described in the Report on " Pipe Line Coefficients," [1] in which the range of velocities was

Fig. 7.



Experimental Data on Wrought-Iron Pipes.

15 to 1. Other carefully conducted experiments include those on the Manchester, Thirlmere siphons (44 inches in diameter), the Sudbury conduit (48 inches in diameter), and the 61-inch diameter siphon experimented on by Fitzgerald.

Practically all of the available data on wrought-iron pipes were obtained by J. R. Freeman. Extreme care was exercised in making the experiments which covered a wide range of velocities. The pipes were considered to be fairly representative of ordinary lap-welded wrought-iron pipes used in the U.S.A. The remaining experiments by J. B. Francis and H. Smith, Jr., indicate that their pipes were considerably smoother than those used by

[1] Issued by a Committee of the New England Water Works Association in 1935: Journal New England Water Works Assoc., vol. 49 (1935).

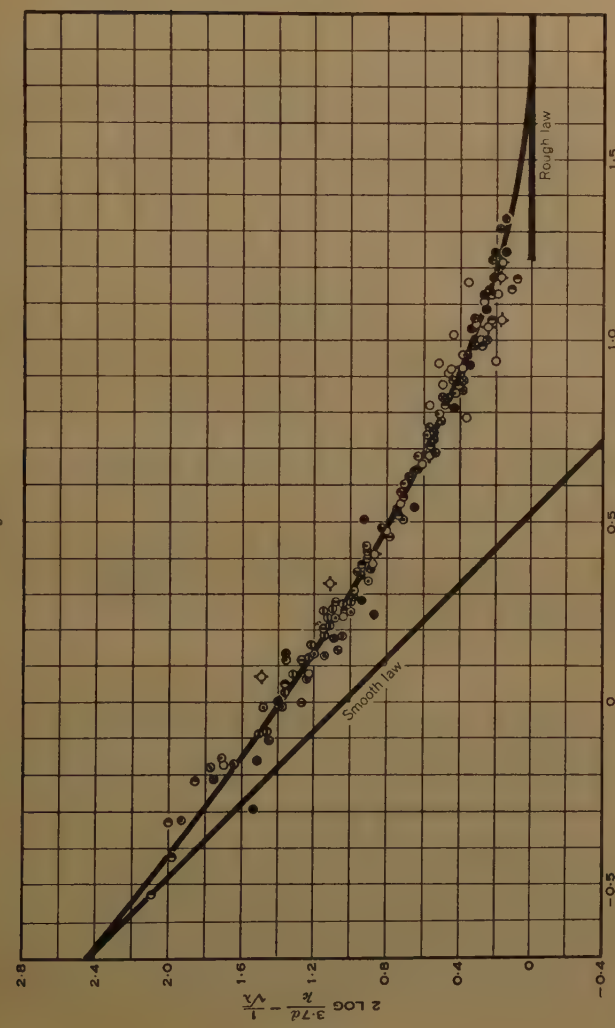
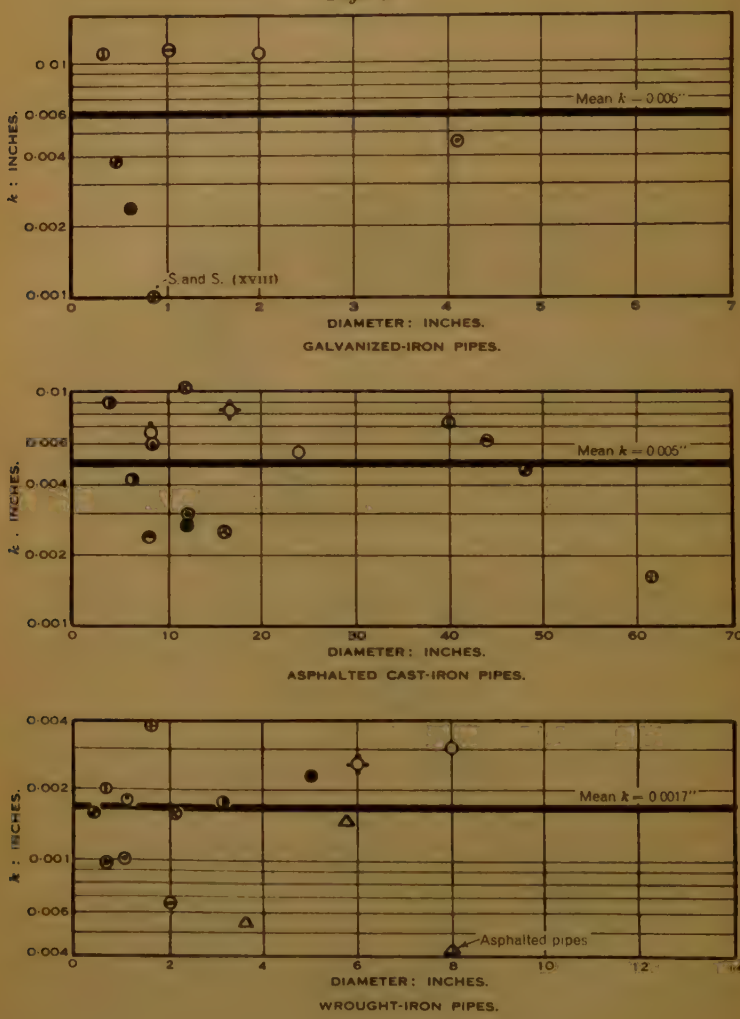


Fig. 8.

Deviations from Rough-Law for Wrought-Iron Pipes.

Freeman, although Freeman's results are remarkably consistent among themselves. Some experiments [1] on asphalted wrought-iron pipes are also

Figs. 9.



included, but these pipes appear to have a capacity averaging about 5 per cent. greater than that of uncoated pipes.

[1] Pipes Nos. 302, 304, and 310 in "The Flow of Water in Riveted Steel and Anagolous Pipes," by F. C. Scobey (U.S. Dept. Agriculture—Tech. Bul. No. 50, Jan. 1930). Denoted in *Fig. 9* of the present Paper by Δ.

The mean values of k are :

Galvanized-iron pipes	$k = 0{\cdot}006$ inch.
Asphalted cast-iron pipes . . .	$k = 0{\cdot}005$ inch.
Uncoated cast-iron pipes . . .	$k = 0{\cdot}01$ inch.
Wrought-iron pipes	$k = 0{\cdot}0017$ inch.

Old Pipes.

The deterioration of pipes with age has already been discussed at some length in a previous Paper [1] so only brief reference to this problem will be made here.

The hydraulic resistance of water-mains increases after the mains have been in service for some time due to growths or deposits upon the internal surfaces. By making various simplifying assumptions it has been possible to develop a formula [1] which gives the relation between the age of a pipe and its carrying capacity, which may be written as

$$\frac{Q}{Q_0} = -\frac{1}{p_0}\log\left(\frac{T\alpha}{3{\cdot}7d} + 10^{-p_0}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad (21)$$

where Q denotes the discharge at the end of T years, Q_0 denotes the initial discharge, $p_0 = C_0/2\sqrt{8g}$ (where C_0 is the initial Chezy coefficient) and α is the average rate of growth of roughness.

If in any district the growth-rate α is required this may be computed from the results of experimental observations by means of the equation

$$\alpha = \frac{3{\cdot}7d}{T}(10^{-p} - 10^{-p_0}) \quad . \quad . \quad . \quad . \quad . \quad (22)$$

where $p = C/2\sqrt{8g}$ and C denotes the final Chezy coefficient.

The diameter of a proposed pipe may be determined from the formula

$$d^{\frac{5}{2}} = 1{\cdot}2\frac{Q}{\sqrt{gi}}\left[\log\frac{Q}{\sqrt{gi}}\frac{14}{(\alpha T + k_0)^{\frac{5}{2}}}\right]^{-1} \quad . \quad . \quad . \quad . \quad (23)$$

where i denotes the hydraulic gradient and k_0 denotes the original roughness size, say 0·01 inch. Alternatively, the appended design-Tables II–VI may be used to determine Chezy coefficients and values of $AC\sqrt{m}$ corresponding to various values of k and d. The roughness k is readily obtained from

$$k = k_0 + \alpha T,$$

and α may be computed from experimental observation using formula (22).

Where no experimental data is available for calculating the growth-rate

[1] C. F. Colebrook and C. M. White, "The Reduction of Carrying Capacity of Pipes with Age." Journal Inst. C.E., vol. 7 (1937–38), p. 99. (November 1937).

α this may be estimated for asphalted cast-iron pipes from the pH value of the water, using the interpolation formula

$$2 \log \alpha = 3{\cdot}8 - p\text{H} \quad . \quad . \quad . \quad . \quad . \quad . \quad (24)$$

which gives the growth-rate in inches per year.

Discussion and Conclusions.

The present analysis of the problem of flow in commercial pipes has been based on the premise that transition from smooth-law to rough-law flow in commercial pipes takes place in a gradual manner, as shown in *Fig. 1* (p. 136). By an extension of the Prandtl–von-Karman laws for smooth and rough pipes, a theoretical transition law (12) has been developed by the Author, in collaboration with Dr. C. M. White, which gives favourable support to this assumption. Although the available experimental data is so incomplete and limited in range that fully rough conditions were only reached in a few cases, a collection of data on old mains shown in *Fig. 2* of a previous Paper [1] proves conclusively that in the case of non-uniformly roughened pipes (which include most commercial pipes), the resistance-coefficient falls with decreasing rapidity as the velocity increases, and once having reached square-law it remains constant at all higher velocities.

The fact that there are considerable variations in the roughness and transition curves in each class of pipe must not be considered a defect in the method of analysis. Such variations are to be expected, since manufacturing conditions are not identical in different plants. For design purposes a series of transition curves for each class is obviously impracticable, so mean curves corresponding to average conditions have been determined. The scatter of the k-values in *Fig. 9* is too great to be able to ascertain any possible dependence of k on pipe-size, so a single value for each class seems justified especially as pipes of all sizes in any particular class are made by the same process. In the case of built-up pipes, such as riveted steel pipes, a variation of k with pipe-size would be expected, and in a later Paper it will be shown that this occurs in the case of a certain class of riveted pipe.

Where it is not possible to determine by experiment the transition curve for any particular type of pipe, the theoretical transition curve (12) may be used with very little error provided that the roughness can be determined, and this is not difficult since some reliable experimental data on a few pipes over at least a small range of velocities is usually available.

All formulas in the Paper are non-dimensional throughout and it is possible, therefore, to use the results in any system of units. Since the transition curves are somewhat complex and are not, therefore, easy to use, five design-Tables (Tables II–VI) based on these functions are included

[1] Footnote (1), p. 153.

in order to facilitate calculations on the flow of water. The Chezy coefficient C in $U = C\sqrt{mi}$ is given for various pipe-sizes, velocities, and gradients in English units at a temperature of 55° F., as calculations involving the Chezy formula are easily and rapidly made by slide-rule. Similar tables for gas, air and other fluids may be compiled by means of the transition curve determined by the Author.

The work was carried out in the Civil Engineering Department of the Imperial College of Science and Technology, London, and the Author is indebted to the generosity of the Clothworkers Company, who, in supporting another research of purely academic nature, indirectly inspired the present work.

The Paper is accompanied by nine sheets of drawings and five design-Tables—from which the Figures in the text and the following Appendixes have been prepared.

APPENDIX.

Examples illustrating the use of the design-Tables.

Problem (*1*).

To find the discharge of a new asphalted cast-iron pipe, 48 inches diameter, with a gradient of 1 in 6,000.

The discharge is determined from

$$Q = (AC\sqrt{m})\sqrt{i}$$

and from Table IV the value of $AC\sqrt{m}$ corresponding to a gradient of the order 1 in 6,000 is

$$AC\sqrt{m} = 1{,}710.$$

Hence

$$Q = 1{,}710 \times \sqrt{0{\cdot}0001667}$$
$$= 22{\cdot}1 \text{ cusecs.}$$

Problem (*2*).

To find the diameter of a new asphalted cast-iron pipe to discharge 10 cusecs with a gradient of 1 in 400.

The size of pipe is determined by the value of

$$AC\sqrt{m} = \frac{Q}{\sqrt{i}} = \frac{10}{\sqrt{0{\cdot}0025}} = 200.$$

From Table IV it is seen that a 21-inch diameter pipe has a value of $AC\sqrt{m} = 208$ at approximately the given gradient.

The actual discharge of this pipe at the given gradient is

$$Q = 208 \,.\, \sqrt{0{\cdot}0025} = 10{\cdot}4 \text{ cusecs}$$

Problem (3).

To find the diameter of an asphalted cast-iron pipe which will discharge 36 cusecs 30 years hence with a gradient of 1 in 100 and a pH value of 7·2.

The required pipe must have a value of

$$AC\sqrt{m} = \frac{Q}{\sqrt{i}} = \frac{36}{\sqrt{0{\cdot}01}} = 360$$

and by interpolation in Table VI for a pH value of 7·2 it is seen that a 33-inch diameter pipe has a value of $AC\sqrt{m} = 365$ approximately at this pH value.

TABLE II.—SMOOTH PIPES: VALUES OF C IN $U = C\sqrt{mi}$ AND $Q = (AC\sqrt{m})\sqrt{i}$ AT VARIOUS VELOCITIES.

D: inches.	A: square feet.	$\sqrt{m}$: (feet)$^{\frac{1}{2}}$.	$U=0{\cdot}5$		$U=0{\cdot}7$		$U=1$		$U=1{\cdot}5$		$U=2$		$U=3$		$U=5$		$U=7$		$U=10$		$U=15$		$U=20$		$U=30$		
			C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	
1	0·00546	0·1444	77·5	0·061	81·5	0·0642	86	0·0678	90·5	0·0713	94	0·074	99	0·078	105	0·0828	110	0·0868	114	0·09	119	0·094	122	0·0965	128	0·101	
2	0·0218	0·2042	86	0·385	89·5	0·4	94	0·42	99	0·44	103	0·46	107·5	0·48	114	0·508	118	0·525	122	0·545	128	0·57	131	0·585	136	0·605	
3	0·0491	0·25	90·5	1·11	94·5	1·16	99·5	1·22	104	1·275	107·5	1·32	112·5	1·38	119	1·46	123	1·51	128	1·57	132·5	1·625	136·5	1·675	141·5	1·735	GRADIENT = $\frac{1}{1}$
4	0·0874	0·289	94	2·37	98	2·47	102·5	2·59	107·5	2·71	111	2·8	116	2·93	122	3·08	126·5	3·19	131	3·31	136	3·43	140	3·53	145	3·66	
5	0·1362	0·323	96·5	4·25	101	4·45	105·5	4·65	110	4·85	113·5	5	119	5·25	125	5·5	129·5	5·7	134·5	5·92	139·5	6·15	143	6·3	148	6·5	
6	0·1965	0·353	99	6·87	103	7·15	107·5	7·45	112·5	7·8	116	8·05	121	8·4	127·5	8·85	131·5	9·12	136·5	9·45	141·5	9·8	145	10·05	150	10·4	
7	0·267	0·382	101	10·3	105	10·7	109·5	11·15	114	11·6	118	12·05	123	12·55	130	13·25	133·5	13·6	138	14·05	143	14·6	147	15·0	151·5	15·45	
8	0·349	0·408	102·5	14·6	106·5	15·2	111	15·8	116	16·5	120	17·1	125	17·8	131	18·7	135·5	19·3	140	20	145	20·6	149·5	21·3	153	21·8	
9	0·442	0·433	104	20	108	20·7	112·5	21·5	117·5	22·5	121	23·2	126	24·1	132·5	25·4	137	26·2	141·5	27·1	146·5	28·1	150·5	28·8	155	29·7	
10	0·546	0·457	105·5	26·3	109·5	27·3	114	28·4	119	29·7	122·5	30·5	127·5	31·8	134	33·4	138·5	34·5	142·5	35·5	148	36·9	152	37·9	156·5	39	
11	0·66	0·478	106·5	33·6	110·5	34·9	115	36·4	120	38	124	39·2	129	40·7	135·5	42·8	139·5	44·1	144	45·5	149	47·1	153	48·3	158	50	
12	0·786	0·5	107·5	42·2	112	44	116	45·5	121·5	47·8	125	49·1	130	51·1	136·5	53·6	140·5	55·3	145	57	150	59	154	60·5	159·5	62·7	
15	1·228	0·56	110·5	76	114·5	78·8	119	81·8	124	85·3	127·5	87·7	132·5	91	139·5	96	143·5	98·7	148	102	153·5	105·5	156·5	107·5	162	111·5	
18	1·767	0·612	112·5	121·5	117	126·5	121	130·5	126	136	130	140·5	135	146	141	152	146	157·5	150	162	155	167·5	159	172	164·5	178	
21	2·407	0·661	114·5	182	119	189	123	196	127·5	203	131·5	209	136·5	217	143	227	148	235	152·5	242	157·5	250	161	256	166	264	
24	3·14	0·707	116	257	120·5	267	125	277	130	288	133·5	296	138·5	307	145	322	150	333	154·5	343	159·5	354	163	362	168	373	
27	3·98	0·75	117·5	351	121·5	363	126	376	131·5	392	135	402	140·5	419	146·5	437	150·5	450	155·5	465	161	480	165	492	170	507	
30	4·91	0·79	119	462	123	477	127·5	495	132·5	515	136	527	141	547	148	575	152	590	156·5	607	162	628	166	642	170·5	661	
33	5·94	0·829	120	592	124	611	128·5	633	134	660	137·5	678	142·5	703	149	735	153·5	757	158	780	163	805	167	823	172	848	GRADIENT = $\frac{1}{10}$
36	7·075	0·866	121·5	745	125	765	130	795	135	825	138·5	848	143·5	880	150	920	154·5	945	159·5	975	164·5	1006	168	1030	173	1060	
40	8·73	0·913	122·5	977	127	1010	131·5	1046	136	1085	140	1115	145	1155	151·5	1206	156	1245	160·5	1280	165·5	1320	169·5	1350	174·5	1390	
44	10·56	0·957	124	1250	128	1290	132·5	1340	137·5	1390	141	1425	146·5	1480	153	1545	157	1585	161·5	1630	167	1685	170·5	1720	176	1780	
48	12·56	1·0	125	1570	129·5	1625	133·5	1680	138·5	1740	142	1785	147·5	1855	154·5	1940	158·5	1990	163	2050	168	2110	171·5	2160	177	2220	
54	15·9	1·061	126·5	2140	131	2210	135	2280	140	2360	143·5	2420	149	2510	156	2630	160	2700	164·5	2780	169·5	2860	173	2920	179	3020	
60	19·64	1·118	127·5	2795	132	2890	136·5	2990	141·5	3100	145	3180	150	3290	157	3440	161·5	3540	165·5	3630	171	3750	174·5	3830	180	3940	
66	23·78	1·172	129	3600	133	3710	137·5	3840	142·5	3980	146·5	4090	151·5	4220	158	4400	162	4520	167	4660	172	4800	176	4910	181·5	5070	
72	28·3	1·225	130·5	4530	134·5	4670	138·5	4810	143·5	4980	147·5	5130	153	5310	159·5	5550	163·5	5680	168	5830	173	6000	177	6150	182·5	6340	
78	33·2	1·275	131	5550	135·5	5730	139·5	5900	145	6150	148·5	6280	154	6520	160·5	6800	164·5	6970	169	7150	174	7380	178	7550	183·5	7770	
84	38·5	1·322	131·5	6700	136	6920	141	7170	146	7420	150	7620	154·5	7850	161	8200	165·5	8420	170	8650	175	8900	179	9100	185	9400	

GRADIENT = $\frac{1}{100,000}$ GRADIENT = $\frac{1}{10,000}$ GRADIENT = $\frac{1}{1,000}$ GRADIENT = $\frac{1}{100}$

TABLE III.—NEW GALVANIZED-IRON PIPES: VALUES OF C IN $U = C\sqrt{mi}$ AND $Q = (AC\sqrt{m})\sqrt{i}$ AT VARIOUS VELOCITIES.

D: inches.	A: square feet.	$\sqrt{m}$: (feet)$^{\frac{1}{2}}$.	$U=0{\cdot}5$		$U=0{\cdot}7$		$U=1$		$U=1{\cdot}5$		$U=2$		$U=3$		$U=5$		$U=7$		$U=10$		$U=20$		$U=30$		
			C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	
0·5	0·00136	0·102	62	0·0086	64·5	0·009	67	0·0093	70	0·0097	71·5	0·0099	73·5	0·0102	76	0·0106	77·5	0·0108	78·5	0·0109	80	0·0111	80	0·0111	
1	0·00546	0·1444	70·5	0·0555	73	0·0575	75·5	0·0595	78·5	0·062	80·5	0·0635	83	0·0655	85	0·067	86·5	0·068	88	0·0695	89·5	0·0705	90	0·071	GRADIENT=
1·5	0·01228	0·1772	75	0·163	78	0·169	81	0·176	84	0·182	85·5	0·185	88	0·191	90·5	0·196	92	0·20	93	0·202	94·5	0·205	95	0·206	10
2	0·0218	0·2042	79	0·352	82	0·365	84·5	0·376	87·5	0·39	89·5	0·398	92	0·41	94·5	0·42	96	0·427	97	0·432	99	0·44	99·5	0·443	1
2·5	0·0341	0·2282	82	0·638	84·5	0·657	87·5	0·68	90·5	0·705	92·5	0·72	95	0·74	97·5	0·76	99	0·77	100	0·78	102	0·795	102·5	0·8	
3	0·0491	0·25	84	1·03	87	1·07	90	1·105	93	1·14	95	1·165	97	1·19	100	1·23	101	1·24	102 5	1·26	104·5	1·28	105	1·29	
3·5	0·0668	0·27	86	1·55	88·5	1·59	91·5	1·65	95	1·71	97	1·74	99	1·78	102	1·84	103	1·85	105	1·89	106·5	1·92	107	1·93	
4	0·0874	0·289	87·5	2·21	90·5	2·29	93·5	2·36	96·5	2·44	99	2·50	101	2·55	103·5	2·62	105	2·65	106·5	2·69	108·5	2·74	109	2·75	
4·5	0·1104	0·306	89	3·01	92	3·11	95	3·21	98	3·31	100	3·38	102·5	3·46	105	3·55	106·5	3·6	108	3·65	110	3·72	110·5	3·74	
5	0·1362	0·323	90·5	3·97	93·5	4·10	96·5	4·23	99·5	4·36	101·5	4·45	104	4·56	106·5	4·67	108	4·75	109·5	4·80	111·5	4·9	112	4·92	
5·5	0·165	0·339	92	5·13	94·5	5·27	97·5	5·45	100·5	5·62	103	5·75	105	5·85	108	6·03	109·5	6·12	111	6·20	113	6·31	113·5	6·33	
6	0·1965	0·353	93	6·45	95·5	6·64	98·5	6·85	101·5	7·05	104	7·22	106	7·37	109	7·58	110·5	7·68	112	7·78	114	7·92	114·5	7·95	
7	0·267	0·382	94·5	9·62	97·5	9·93	100·5	10·25	103·5	10·55	105·5	10·75	108	11·0	111	11·2	112·5	11·5	114	11·6	116	11·8	117	11·9	
8	0·349	0·408	96	13·6	99·5	14·15	102·5	14·6	105·5	15·0	107·5	15·3	110	15·65	113	16·1	114·5	16·3	116	16·5	118	16·8	119	17	GRADIENT=
9	0·442	0·433	97·5	18·7	100·5	19·3	104	19·9	107	20·5	109	20·9	111·5	21·4	114·5	21·9	116	22·2	117·5	22·5	119·5	22·9	120	23	1
10	0·546	0·457	99	24·7	102	25·4	105	26·2	108·5	27	110	27·4	113	28·2	116	28·9	117·5	29·3	119	29·6	120·5	30·0	121·5	30·2	1
11	0·66	0·478	100	31·5	103	32·5	106	33·4	109·5	34·5	111·5	35·2	114	36	117	36·9	119	37·5	120	37·8	122	38·5	123	38·8	
12	0·786	0·50	101	39·6	104	40·8	107·5	42·2	111	43·5	113	44·3	115·5	45·2	118·5	46·5	120	47·1	121·5	47·6	123·5	48·5	124·5	48·8	

GRADIENT= $\frac{1}{10{,}000}$ GRADIENT = $\frac{1}{1000}$ GRADIENT = $\frac{1}{100}$ GRADIENT = $\frac{1}{10}$

Table IV.—New Asphalted Cast-Iron Pipes: Values of C in $U = C\sqrt{m}\sqrt{i}$ and $Q = (AC\sqrt{m})\sqrt{i}$ at Various Velocities.

D: Inches.	A: square feet.	$\sqrt{m}$: (feet)$^{\frac{1}{2}}$.	$U=0{\cdot}5$		$U=0{\cdot}7$		$U=1{\cdot}0$		$U=1{\cdot}5$		$U=2{\cdot}0$		$U=3{\cdot}0$		$U=5{\cdot}0$		$U=7{\cdot}0$		$U=10{\cdot}0$		$U=20{\cdot}0$		$U=30{\cdot}0$		
			C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	
3	0·0491	0·25	88·5	1·08	91·5	1·12	94·5	1·16	98	1·20	99·5	1·22	101·5	1·24	104	1·28	105·5	1·29	106·5	1·31	107·5	1·32	107·5	1·32	GRADIENT =
4	0·0873	0·288	92·5	2·32	95·5	2·40	98·5	2·48	101·5	2·55	103	2·59	105·5	2·65	108	2·71	109·5	2·75	110·5	2·78	111	2·79	111·5	2·80	$\frac{1}{1}$
5	0·136	0·323	95·5	4·18	98·5	4·32	101	4·43	104·5	4·58	106·5	4·67	109	4·78	111	4·87	112·5	4·93	113·5	4·98	114·5	5·03	114·5	5·03	
6	0·196	0·354	98	6·82	100·5	7·00	103	7·16	106·5	7·40	108·5	7·55	111	7·72	113·5	7·90	115	8·0	115·5	8·03	117	8·13	117	8·13	
7	0·267	0·383	99·5	10·17	102·5	10·5	105·5	10·8	108·5	11·1	110·5	11·3	113	11·55	115·5	11·8	116·5	11·9	118	12·05	119	12·2	119	12·2	
8	0·349	0·408	101	14·4	104	14·8	107	15·2	110	15·6	112	15·9	114·5	16·3	117	16·7	118·5	16·9	119·5	17·0	120·5	17·1	121	17·2	
9	0·442	0·433	102·5	19·6	106	20·3	109	20·9	112	21·5	114	21·8	116·5	22·3	119	22·8	120	23	121·5	23·3	122	23·4	123	23·6	
10	0·545	0·457	104	25·9	107·5	26·8	110	27·4	113·5	28·3	115·5	28·8	118	29·4	120·5	30	121·5	30·3	123	30·7	124	30·9	124·5	31	
11	0·66	0·478	105·5	33·2	108·5	34·2	111·5	35·2	114·5	36·1	116·5	36·7	119	37·5	121·5	38·3	123	38·8	124	39·1	125·5	39·6	125·5	39·6	
12	0·785	0·5	106·5	41·8	109·5	42·9	112·5	44·1	116	45·5	118	46·2	120·5	47·2	123	48·2	124·5	48·8	125·5	49·2	127	49·8	127	49·8	
15	1·227	0·56	109·5	75·4	112	77·1	115·5	79·5	119	82·0	121	83·3	123·5	85	126	86·7	127·5	87·8	128·5	88·5	130	89·5	130	89·5	
18	1·767	0·613	111·5	121	114·5	124	117·5	127·5	121	131	123	133·5	125·5	136	128·5	139	130	141	131	142	132·5	144	132·5	144	GRADIENT =
21	2·405	0·662	113·5	181	116·5	186	120	191	123	196	125	199	127·5	203	130·5	208	132	210	133	212	134·5	214	134·5	214	$\frac{1}{10}$
24	3·14	0·707	115	255	118·5	263	121·5	270	125	278	127	282	129·5	288	132·5	294	133·5	296	135	300	136·5	303	136·5	303	
27	3·98	0·75	116·5	347	120	358	123	366	126·5	377	128·5	383	131	390	134	400	135·5	403	136·5	407	137·5	410	138	412	
30	4·91	0·79	118	458	121	470	124·5	483	128	497	130	505	132·5	515	135	524	136·5	530	138	535	139	540	139·5	542	
33	5·94	0·83	119	587	122	602	126	622	129	637	131	647	134	662	136	672	138	681	139·5	689	140·5	694	141	696	
36	7·07	0·866	120	735	123·5	756	127	777	130	797	132	808	135	827	137·5	842	139	852	140·5	860	142	870	142	870	
40	8·73	0·912	121·5	967	125	995	128	1018	131·5	1045	133·5	1061	136·5	1087	139	1105	140·5	1116	142	1130	143	1136	143·5	1140	
44	10·56	0·955	123	1240	126	1270	129·5	1305	132·5	1335	135	1360	137·5	1385	140	1415	142	1430	143	1440	144·5	1455	145	1460	
48	12·57	1·0	124	1560	127·5	1605	130·5	1640	134	1685	136	1710	139	1750	141·5	1780	143	1800	144·5	1820	146	1835	146	1835	
54	15·90	1·06	125·5	2120	129	2180	132	2230	135·5	2290	137·5	2320	140	2365	143	2415	144·5	2440	146	2470	148	2490	147·5	2490	
60	19·63	1·12	126·5	2780	130	2860	133·5	2940	137	3020	139	3060	141·5	3115	145	3180	146	3210	148	3250	149	3280	149·5	3290	

GRADIENT = $\frac{1}{10{,}000}$ | GRADIENT = $\frac{1}{1{,}000}$ | GRADIENT = $\frac{1}{100}$

[Table V.

TABLE V.—NEW WROUGHT-IRON PIPES: VALUES OF C IN $U = C\sqrt{mi}$ AND $Q = (AC\sqrt{m})\sqrt{i}$ AT VARIOUS VELOCITIES.

D: inches.	A: square feet.	$\sqrt{m}$: (feet)½.	U=0·5		U=0·7		U=1		U=1·5		U=2		U=3		U=5		U=7		U=10		U=15		U=20		U=30		
			C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	
0·5	0·001362	0·102	67·5	0·0094	70·5	0·0098	74	0·0103	77	0·0107	79·5	0·0111	83	0·0115	86·5	0·012	88·5	0·0123	90·5	0·0126	92·5	0·0129	94	0·0131	95·5	0·0133	GRADIENT=
1	0·00546	0·1445	76	0·060	79	0·0622	82	0·065	85·5	0·0675	88	0·0695	91·5	0·072	95	0·075	97·5	0·077	99·5	0·0785	101·5	0·080	103	0·081	104·5	0·0825	10 / 1
1·5	0·01228	0·1772	80·5	0·175	84	0·182	87·5	0·19	91	0·197	93·5	0·203	96·5	0·209	100·5	0·218	102·5	0·222	105	0·228	107	0·232	108·5	0·235	110	0·239	
2	0·02182	0·204	84	0·374	87·5	0·389	91	0·405	94·5	0·421	97	0·432	100·5	0·448	104	0·463	106·5	0·475	109	0·485	111	0·495	112·5	0·50	114	0·507	
2·5	0·0341	0·228	87	0·677	90	0·70	93·5	0·727	97·5	0·758	100	0·778	103	0·802	107	0·833	109·5	0·852	111·5	0·867	114	0·887	115·5	0·90	117	0·91	
3	0·0491	0·25	89·5	1·10	92·5	1·14	96	1·18	100	1·23	102	1·25	105·5	1·30	109·5	1·34	112	1·37	114	1·40	116·5	1·43	118	1·45	119·5	1·47	
3·5	0·0668	0·27	91·5	1·64	94·5	1·70	98	1·76	101·5	1·83	104	1·87	107·5	1·93	111	2·00	113·5	2·04	116	2·09	118·5	2·13	120	2·16	121·5	2·18	
4	0·0874	0·289	93	2·35	96	2·42	99·5	2·51	103	2·60	105·5	2·66	109	2·75	113	2·85	115·5	2·92	118	2·98	120	3·03	122	3·08	123·5	3·12	
4·5	0·1104	0·306	94·5	3·20	98	3·31	101	3·41	104·5	3·53	107	3·62	110·5	3·73	114·5	3·87	117	3·96	119·5	4·03	121·5	4·10	123·5	4·18	125	4·22	
5	0·1362	0·323	96	4·20	99·5	4·36	102·5	4·49	106	4·65	108·5	4·75	112	4·91	116	5·08	118·5	5·20	121	5·30	123·5	5·41	125	5·48	126·5	5·55	
5·5	0·165	0·339	97	5·42	100·5	5·61	103·5	5·78	107·5	6·00	110	6·15	113·5	6·33	117·5	6·55	120	6·70	122·5	6 84	125	6·98	126	7·03	128	7·15	
6	0·1965	0·353	98	6·82	101·5	7·05	105	7·30	108·5	7·55	111	7·72	114·5	7·96	118·5	8·23	121	8·40	123·5	8·58	126	8·75	127·5	8·87	129	8·97	GRADIENT=
7	0·267	0·383	100	10·2	103	10·5	106·5	10·85	110·5	11·3	113	11·5	116·5	11·9	120	12·25	122·5	12·5	125	12·75	127·5	13·0	129	13·2	130·5	13·3	1 / 1
8	0·349	0·408	101·5	14·4	104·5	14·8	108	15·4	112	15·9	114·5	16·3	118	16·8	122	17·3	124	17·6	126·5	18·0	129·5	18·4	130·5	18·6	132·5	18·8	
9	0·442	0·433	103	19·7	106	20·3	109·5	21·0	113·5	21·7	116	22·2	119·5	22·9	123·5	23·7	126	24·1	128·5	24·6	131	25·1	132·5	25·4	134·5	25·8	
10	0·546	0·457	104·5	26·0	107·5	26·8	111	27·7	115	28·6	117·5	29·3	121	30·1	125	31·1	127·5	31·8	130	32·4	132·5	33·0	134	33·4	136	33·9	
11	0·66	0·478	105·5	33·2	109	34·3	112·5	35·4	116	36·6	119	37·5	122·5	38·6	126	39·8	128·5	40·5	131	41·3	133·5	42·1	135	42·6	137	43·2	
12	0·786	0·5	106·5	41·7	110	43·1	113·5	44·5	117	45·8	120	47·0	123·5	48·4	127	49·8	129·5	50·8	132	51·7	135	52·9	136	53·3	138	54·1	
			GRADIENT = 1/10,000				GRADIENT = 1/1,000				GRADIENT = 1/100						GRADIENT = 1/10										

TABLE VI.—OLD PIPES: VALUES OF C IN $V = C\sqrt{m}\sqrt{i}$ AND $Q = (AC\sqrt{m})\sqrt{i}$ FOR VARIOUS VALUES OF k.

D: inches.	A: square feet.	$\sqrt{m}$: (feet)$^{\frac{1}{2}}$.	k=0·05 inch.		k=0·075 inch.		k=0·1 inch.		k=0·15 inch.		k=0·2 inch.		k=0·3 inch.		k=0·5 inch.		k=0·75 inch.		k=1·0 inch.		k=1·5 inch.		k=2·0 inch.		k=3·0 inch.	
			C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$	C	$AC\sqrt{m}$
3	0·0491	0·25	75 3	0·92	69·6	0·85	65·7	0·81	60	0·74	56	0·69	50·3	0·62	43·2	0·53	37·6	0·46	33·6	0·41	27·9	0·34	23·9	0·29	18·2	0·22
4	0·0873	0·288	79·3	2·0	73·7	1·86	69·6	1·76	64	1·61	60	1·51	54·4	1·37	47·2	1·19	41·6	1·05	37·6	0·95	31·9	0·8	27·9	0·7	22·3	0·56
5	0·136	0·322	82·5	3·63	76·8	3·38	72·8	3·2	67·2	2·96	63·2	2·78	57·5	2·53	50·3	2·21	44·7	1·96	40·7	1·79	35·1	1·54	31	1·36	25·4	1·11
6	0·196	0·354	85	5·90	79·4	5·52	75·3	5·23	69·7	4·85	65·7	4·57	60	4·17	52·9	3·68	47·2	3·28	43·2	3·00	37·6	2·61	33·5	2·33	27·9	1·94
7	0·267	0·383	87·2	8·90	81·5	8·30	77·4	7·9	71·8	7·33	67·8	6·92	62·2	6·35	55	5·61	49·4	5·06	45·4	4·65	39·8	4·08	35·7	3·64	30·1	3·07
8	0·349	0·408	89	12·7	83·3	11·8	79·3	11·3	73·7	10·5	69·7	9·9	64·1	9·12	56·9	8·1	51·3	7·3	47·2	6·72	41·6	5·92	37·6	5·35	31·9	4·54
9	0·442	0·433	90·7	17·4	85	16·3	80·8	15·5	75·3	14·4	71·2	13·6	65·7	12·6	58·5	11·2	52·9	10·1	48·8	9·35	43·2	8·30	39·2	7·50	33·6	6·45
10	0·545	0·457	92·1	22·9	86·4	21·5	82·5	20·5	76·8	19·1	72·8	18·1	67·2	16·7	60	14·9	54·3	13·5	50·3	12·5	44·7	11·1	40·7	10·1	35·1	8·75
11	0·66	0·478	93·5	29·5	87·8	27·6	83·8	26·4	78·2	24·7	74·2	23·4	68·5	21·6	61·3	19·4	55·7	17·6	51·7	16·3	46·1	14·5	42·1	13·3	36·4	11·5
12	0·785	0·5	94·7	37·1	89	34·9	85	33·2	79·3	31·1	75·3	29·5	69·7	27·3	62·5	24·5	56·9	22·3	52·9	20·7	47·2	18·5	43·2	16·9	37·6	14·75
15	1·227	0·56	97·8	67·2	92·1	63·3	88·2	60·7	82·5	56·8	78·3	53·9	72·8	50·0	65·7	45·2	60	41·3	56	38·5	50·3	34·6	46·3	31·9	40·7	28·0
18	1·767	0·613	100·3	109	94·6	102	90·7	98·0	85	92·0	81	87·5	75·4	81·5	68·2	74·0	62·5	67·5	58·6	63·3	52·9	57·2	48·8	52·7	43·2	46·7
21	2·405	0·662	102·4	163	96·7	154	92·8	148	87·1	139	83·1	132	77·5	123	70·3	112	64·7	103	60·7	96·7	55	87·7	51·1	81·5	45·3	72·2
24	3·14	0·707	104·3	232	98·7	219	94·7	210	89	198	85	189	79·3	176	72·2	160	66·7	148	62·6	139	56·9	126	52·9	117	47·2	105
27	3·98	0·75	105·9	316	100·2	299	96·3	287	90·6	270	86·7	258	81	241	73·7	220	68·2	203	64·2	191	58·6	175	54·6	163	48·8	145
30	4·91	0·79	107·5	417	101·7	395	97·8	379	92·1	357	88	341	82·5	320	75·3	292	69·7	271	65·7	255	60	233	56	217	50·3	195
33	5·94	0·83	108·7	537	103	508	99	488	93·5	461	89·3	440	83·8	413	76·7	378	71	350	67	330	61·3	302	57·3	283	51·7	255
36	7·07	0·866	110	673	104·4	640	100·2	614	94·7	580	90·6	555	85	520	77·8	477	72·2	442	68·2	418	62·6	383	58·5	358	52·9	324
40	8·73	0·912	111·3	886	105·8	843	101·6	810	96·2	765	92·1	733	86·5	688	79·3	630	73·7	587	69·7	555	64·1	510	60	477	54·3	432
44	10·56	0·955	112·7	1140	107	1080	103	1040	97·5	983	93·3	940	87·8	885	80·6	813	75	757	71	716	65·3	658	61·3	619	55·7	562
48	12·57	1·0	114	1432	108·3	1360	104·4	1310	98·6	1240	94·6	1190	89	1120	81·9	1030	76·2	957	72·2	907	66·5	837	62·5	785	56·9	715
54	15·90	1·06	115·6	1950	110	1860	106	1790	100·4	1700	96·2	1624	90·7	1530	83·5	1410	77·8	1314	73·8	1245	68·3	1152	64·2	1084	58·5	987
60	19·63	1·12	117	2570	111·4	2450	107·4	2360	101·9	2240	97·7	2150	92·2	2030	85	1870	79·3	1745	75·3	1660	69·7	1532	65·7	1445	60	1320
66	23·76	1·17	118·5	3290	112·9	3140	108·8	3020	103·2	2870	99	2750	93·5	2600	86·3	2400	80·7	2245	76·7	2130	71	1975	67	1860	61·3	1705
72	28·27	1·225	119·8	4150	114	3960	110	3820	104·4	3620	100·2	3480	94·8	3290	87·6	3040	81·9	2840	77·9	2700	72·3	2510	68·2	2370	62·6	2170
78	33·18	1·275	120·8	5120	115·1	4880	111·1	4710	105·5	4470	101·4	4300	95·8	4060	88·7	3760	83	3520	79	3350	73·3	3110	69·3	2940	63·7	2700
84	38·48	1·325	121·8	6200	116	5900	112·1	5710	106·5	5430	102·4	5220	96·8	4930	89·7	4570	84	4270	80	4070	74·3	3780	70·3	3580	64·7	3290
*p*H* value of water to give above values of k after 30 years' growth in cast-iron pipes			9·5		9·1		8·8		8·4		8·2		7·8		7·4		7·0		6·8		6·4		6·2		5·8	

* Based on $2 \log a = 3{\cdot}8 - p\text{H}$, where a denotes the growth-rate in inches per year $= \dfrac{k'' - 0{\cdot}005''}{30}$.

Paper No. 5190.

"Anti-Malarial Operations in the Delhi Urban Area."

By Arthur William Henry Dean, M.C., B.Sc., M. Inst. C.E.

(*Abridged*).[1]

(*Ordered by the Council to be published with written discussion.*)[2]

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Introduction.

The Paper describes the geographical and physical features of the area, including the Jumna river and the ridge, a spur of the Aravali hills, which largely determine its configuration, and the various drainage-channels—both natural and man-made (the Western Jumna canal)—and the interruptions to natural drainage (the Grand Trunk road and the Delhi–Agra railway). Details are given of the distribution of population and of the Governmental and Municipal organizations through which the area is administered.

The Propagation and Incidence of Malaria in Delhi.

General.

The malaria parasite is carried from man to man by the bites of anopheline mosquitoes. The parasite can only develop, however, in certain species of anophelines, and the majority are therefore harmless as regards the transmission of malaria. Male mosquitoes are not concerned in the spread of the disease, because their proboscis is not able to pierce the skin, and therefore they cannot obtain a blood meal. Their normal food consists chiefly of fruit and vegetable juices.

[1] The MS. and illustrations may be seen in the Institution Library.—Sec. Inst. C.E.

[2] Correspondence on this Paper can be accepted until the 15th May, 1939, and will be published in the Institution Journal for October 1939.—Sec. Inst. C.E.

Malaria cannot be transmitted from man to man by the mosquito until the parasite has undergone a cycle of development within it, which takes about 10 days to complete. Similarly, the parasite cannot survive in the mosquito unless it has arrived at a particular stage in its development before the insect takes its blood meal. This is reached about 10 days after the first appearance of fever-symptoms, and about 20 days after the patient has received his infective bite.

It is thus evident that a mosquito cannot transmit malaria unless it has taken two human-blood meals, with an interval between them of not less than 10 days. The transmission of the disease is therefore largely dependent on the length of life of the mosquito and on its avidity for human blood.

This is the reason why the malaria season in northern India is limited to a comparatively short period of the year. It is only when the temperature is continuously favourable, and the atmospheric humidity uniformly high throughout the 24 hours, that mosquitoes can survive long enough to transmit malaria. They become sluggish and rarely bite during the winter months, and the rate of development of the parasite within them is greatly retarded.

In Delhi Province the rainfall, which averages 26·84 inches per annum, is due to two distinct agencies—western disturbances and monsoon depressions. During the period from November to May a series of from four to six disturbances enter north-west India from the west each month. These cause rain during winter and dust-storms, sometimes accompanied by rain, in the hot weather. In the rainy season depressions originating at the head of the Bay of Bengal travel along the Gangetic plain in a westerly or north-westerly direction, and cause occasional local heavy rain and general over-cast skies. Conditions become favourable for mosquito-breeding about the end of March, when mosquitoes of the culicine species show an enormous increase, and give rise to the "spring mosquito nuisance." This reaches its height in April and dies away in May. The increase in anopheline mosquitoes occurs rather later, as it begins in April, reaches a maximum during the first fortnight of May, and dies away in June. Luckily it is too dry during these months for mosquitoes to live long, and hence most of them die before they can transmit malaria. During the rainy season there is a second rise in the number of mosquitoes, but this time the anopheline species predominate. Culicines also breed at this time of the year and include those known as *Stegomyia*, which are a particularly unpleasant genus, for they bite fiercely even in day-time.

The most favourable conditions for mosquitoes arise when there is an early long-drawn-out monsoon, with a large total precipitation of rain falling evenly and at frequent intervals without excessive heavy downpours, as these usually wash out shallow breeding places and kill large numbers of mosquito larvæ.

In every year there is a very large autumnal increase in malaria, commencing towards the end of August and ceasing about the third week in October, after which metereological conditions are usually not suitable for mosquitoes to live long enough to allow the malarial parasite to complete its life-cycle.

In years where conditions are favourable malaria becomes epidemic. In minor epidemics, such as occurred in 1926 and in 1933, the incidence is at least twice as much as normal, and in a severe epidemic, the last of which affected the southern part of the Punjab, including Delhi, in 1908, the number of deaths can be enormous; it was computed in that year to have exceeded a quarter of a million. It can readily be realized that the effective control of malaria adds enormously to the prosperity of the country.

Breeding of Mosquitoes.

It is essential to study in complete detail the life-history of the various mosquitoes found breeding in any locality, and to determine which of these are malaria-carriers under the conditions prevailing, before any attempt is made to introduce any control-measures. Neglect of this has led in the past to serious waste of money and to the discrediting of malaria control-measures. For instance, in Panama the chief malaria-carrying mosquito breeds in the standing water of ponds and marshes, and the drainage of these and the clearing of jungle were most successful in controlling malaria there. In the hilly parts of Malaya the chief carrier breeds in running water where land has been cleared of jungle, and the conversion of hill streams into a series of pools by means of dams—the exact opposite of the procedure at Panama—has been found to reduce malaria. Again, in Assam the principal breeding place is water exposed to sunshine, and the indiscriminate cutting of jungle caused a vast increase in malaria, now brought under control by growing shade-trees and bushes along all streams and drainage-channels.

A fact which is still not generally known is that the species of anophelines which carry malaria in northern India almost invariably require clean water in which to breed. They do not lay their eggs in foul water, and hence the presence of collections of filthy water in any particular locality, however objectionable this may be in other respects, does not necessarily mean that it is malarious. Indeed, in certain places water collections have been deliberately fouled by throwing manure or rotting vegetation into them as an anti-malarial measure.

These same foul collections of water, however, produce myriads of mosquitoes. These are of the culicine species and are harmless as regards malaria-transmission, but their bites are much more severe than those of anophelines. The chief malaria-carrier of northern India, *Anopheles culicifacies*, is a most unobtrusive insect. It is small and inconspicuous, and its bite produces little or no effect on the majority of people.

The favourite breeding place of *A. culicifacies* is one in which there is a supply of fresh water which is slowly but constantly renewed. Ideal conditions are provided by the grassy edges of irrigation-channels and of swampy areas fed by leakage from them. It is the amount of edge, not the extent of water-surface, which is of importance. Often there are very few mosquito larvæ in large collections of water, possibly on account of wave-action, or because they more easily become a prey to their numerous natural enemies. The larvæ may, however, be found in enormous numbers round the margins of masonry tanks, such as swimming-pools, unless the tanks are regularly emptied out and allowed to become completely dry.

Range of Flight.

The range of flight is another factor of great practical importance in planning anti-larval measures, as it is most necessary to know the size of the area over which control must be practised. Where the food-supply is abundant anophelines will rarely fly farther than is necessary for them to obtain a blood meal, but there are various factors which influence the range of flight. The distance of spread from the breeding place varies directly with the intensity of breeding, and inversely with the density of habitation in the vicinity. The direction of the prevailing wind, the presence or absence of a belt of forest, and the species of anopheles concerned, have also a marked effect, the larger species usually flying greater distances than the smaller. As a general rule it may be stated that the average range of flight rarely exceeds from $\frac{1}{2}$ to $\frac{3}{4}$ mile, but where breeding is profuse, habitations scarce, and the direction of the wind favourable, anophelines may penetrate to distances of from $1\frac{1}{2}$ to 2 miles from their breeding places, or even farther.

In the Delhi urban area it was considered sufficient to lay down a line $\frac{1}{2}$ mile beyond the outer limits of the built-over zone, and to endeavour to make control of breeding places effective up to that line. In certain cases, however, where mosquito-breeding has been particularly intense, it has been found necessary to extend control beyond this line.

It will be noted that what is described throughout has been the control of malaria, and this is all that is possible. It is sometimes said that malaria has been "eradicated" from, say, Panama. Actually in that country it is only held in check by unremitting control, at a cost justified only by the vast commercial interests involved. There, as everywhere, the least relaxation of control is followed by an outbreak of malaria.

In order to apply control-measures economically it is first necessary to determine the relative incidence of malaria in different parts of the area. This is best done by taking a spleen census of children and mapping the results. An invariable result of malarial infection in children is enlargement of the spleen, and a manual examination of the abdomen of

all the children in a locality reveals those who have this defect. The result is tabulated as a percentage. In order to obtain really comparable results, it is desirable that the same medical man should make all the examinations, as recognition of the severity of the condition is a matter of individual judgement.

Special Features of the Delhi Urban Area.

Special features of the Delhi urban area which lend themselves to intensive breeding of malaria-carrying mosquitoes, such as the Bela swamp, the railway borrow-pits, and the tail-distributary of the Western Jumna canal, are described in detail in the Paper. A history of the measures proposed to control malaria in building New Delhi as the capital of modern India is also given.

Anti-Malarial Measures: Recurrent Control.

Catching-Stations.

Before any control-measures are introduced it is necessary to set up catching-stations at various points, in order to gauge the results. In the Delhi urban area twenty-seven such stations were established. These are shelters (usually stables or poor-class houses) which provide suitable resting places for mosquitoes. Each is visited at regular intervals by the same insect-collector, who searches for mosquitoes for a definite period of time. The records obtained from catching-stations afford the best indications of the efficiency of anti-mosquito, and particularly of anti-larval, measures, a rise in the number of mosquitoes caught in any particular station indicating that some breeding-place has been overlooked.

The catching can be done quite easily by using a test-tube, which is placed over the insect as it rests on some object, the mouth being closed by a plug of cotton wool after the insect has flown in. An electric torch is of great assistance.

Control-Measures.

The control-measures carried out are directed against both the adult insect and the larva. The control-measures are of three main forms: traps, fumigation, and sprays. Many varieties of mosquito-traps have been devised, but it cannot be said that any of them has achieved much success as an anti-mosquito measure, although they afford useful evidence of the species of mosquito present in a locality.

Fumigation has been practised, but it has been found, whatever fumigant was used, that it possesses certain disadvantages; namely, that to be effective it must be repeated at frequent intervals (every 3 or 4 days), and that its use involves the evacuation of the rooms treated for several

hours, and also necessitates the removal of all contents liable to damage such as metal-work, watches, etc. Its use has now been dropped in favour of spraying as the standard method of dealing with the adult mosquitoe in living quarters.

In recent years a large number of proprietary sprays have been put on the market, most of them having a basis of kerosene oil. The most active agent is an extract of the pyrethrum plant (*Chrysanthemum cinerariæfolium*). The Malaria Institute of India (formerly the Malaria Survey of India) has introduced a mixture, composed of 1 part of "Pyrocide 20" in 19 parts of kerosene oil, which is more effective as a mosquitocide than any other spray in use. "Pyrocide 20" is an extract of pyrethrum manufactured in England from the plant, which has to be imported from Dalmatia or Japan; hence it is decidedly expensive, costing 35 rupees per gallon free on rail at Indian ports. Even so the cost of a gallon of the spray is only 2 rupees 8 annas, which compares very favourably with all proprietary preparations on the market. Attempts are now being made by the Indian Agricultural Department to cultivate the plant in various parts of India, and if these are successful a considerable reduction in price will follow.

Measures directed against the Larvæ of Mosquitoes.

Permanent engineering works come into this category, and as they lead, when properly designed and maintained, to the complete abolition of breeding-places, they should be adopted wherever possible. Temporary measures are employed where the cause of breeding is temporary (for example during the construction of railways, buildings, etc.), and where the cost of permanent measures is prohibitive; that is, where effective temporary measures can be carried out each year at a recurring cost less than that involved by interest and maintenance-charges on the equivalent permanent works.

The larvicides used are either oils or chemicals. The former owe their success essentially to a mechanical effect, the larvæ being suffocated by producing a surface film which cuts off their supply of air; also, by reducing the surface-tension, they make it difficult for the larvæ to remain on the surface, thus causing them to "drown". Certain varieties claim in addition toxic effects either of the vapour, of the oil direct, or indirectly by the solubility of the oil in water. The chemical larvicides, of which the most important is "Paris Green" (Copper aceto-arsenite containing not less than 50 per cent. of arsenious oxide), act as poisons. Almost every grade of oil on the market is used for mosquito-larvæ destruction under appropriate circumstances. Thus petrol, with 1 per cent. of turpentine added to increase its spreading power, can be used in wells, 24 ounces being used to every 80 square feet of surface. After from 2 to 3 hours there is no smell or taste of petrol in the water and all larvæ are killed. Kerosene oil, or solar oil with a small addition of

some agent to increase its spreading power, such as 1 per cent. of castor oil or cresol, is most suitable on moving water or water with much surface vegetation, where a rapidly-spreading oil with a high toxicity is indicated. Diesel oil and the " liquid fuel " of the petroleum companies (a residue after some of the more volatile constituents have been distilled off) are best used on still water, where after some hours they spread to form a complete film which kills all the larvæ. Waste motor- or crank-case oil with the addition of 10 per cent. of kerosene and from 1 to 2 per cent. of castor oil has also been used, and is both cheap and long lasting, but destroys vegetation. Methods of applying oil are equally varied, but spraying is the most effective for general use. A knap-sack type sprayer, fitted with a pump worked by hand and having a capacity of from 2½ to 3 gallons, is most suitable for use by an Indian coolie. By this means oil can be distributed to a distance of from 20 to 30 feet from the operator. The sprayers should be fitted with leather or fibre valves and flexible metallic hose, as petroleum oils cause rubber to perish. The nozzle should deliver a cone spray about 18 inches in diameter 4 feet in front of it. The oil must be filtered when put into the sprayer, and the apparatus needs washing out with a pint of kerosene after each day's use. Balls or " pillows " of used cotton waste or tow, wrapped in pieces of sacking about 28 inches long and 16 inches broad, which will soak up about 2 gallons of oil initially and about 1 gallon when re-soaked (which is done every 7 or 10 days), are tethered along the edges of streams and irrigation-channels or are placed in drains. The oil oozes out gradually and comes to the surface, and this method is very effective on slow-moving streams. A drip-can is another useful device for oiling drains and streams. About ten drops per minute is suitable for a water-surface 1 foot wide.

For oiling swamps about 15 gallons per acre is generally an ample estimate for initial oiling, whilst about 1 gallon for 100 yards of bank is necessary at the first application to destroy the marginal vegetation for a distance of about 1 foot. Subsequent oilings, which should be repeated every 7 or 10 days during the malaria season, will require about half these quantities.

" Paris Green " has been used as an insecticide for crops for many years, and was suggested as an anopheline larvicide comparatively recently. It should be ground fine enough to pass a 200-mesh screen. For use it has to be mixed in varying proportions with a dry inert powder as a diluent, which acts as the vehicle for its distribution. A large number of substances have been tried, including road dust, slaked lime, powdered soap-stone, cement, and brick dust. For large areas of swamp, etc., when used from rotary blowers or aeroplanes, a light diluent is required, and in India soap-stone or lime not completely slaked have been found most satisfactory. Like the " Paris Green ", the diluent must also be passed through about a 200-mesh screen. For dusting small streams a heavier diluent is required, and road-dust has been used extensively in India. The strength required varies con-

siderably with the method of use. For dusting narrow streams, drains, small pools, etc., using road dust as the diluent and throwing by hand after the manner of sowing grain, 1 per cent. is sufficient. For use with rotary blowers over larger areas not less than 2½ per cent. should be used, and 5 per cent. is usually more suitable, whilst up to 15 per cent. is necessary with a large electric blower-unit mounted on a boat. From an aeroplane up to 33 per cent. is used. Here the powder is fed from a hopper into the constriction of a venturi tube, and at a speed of about 65 miles per hour a lethal path about 200 yards wide is obtained. With 4 hours flying for 5 days per week an area of 20 square miles can be covered. Dilution of "Paris Green" is by volume, and not by weight, as the weights of different diluents vary widely. The quantity averages from ½ to 1 lb. of the undiluted "Paris Green" per acre, and the applications should be made at weekly intervals.

"Paris Green" is cheaper than oiling, and it has the great advantages of neither killing fish nor rendering the water unfit for human and animal consumption, as oiling does. It is, however, rather more complicated in preparation and use, and hence it requires more supervision. In particular, it is impossible by superficial inspection to see if it has been applied properly, whilst this is obvious in the case of oiling. It is therefore necessary to inspect systematically, by having a selected group of points in each breeding-area where a specified number of dips are made with a ladle and the average number of larvæ obtained per dip are counted, the process being repeated 24 hours after dusting.

In addition to these main control-measures, various other items, such as the clearing of vegetation along the banks of streams and irrigation-channels, and around ponds and lakes, also come under the head control and have to be regularly repeated.

Permanent Engineering Control-works.

Permanent engineering methods of control are described in the Paper, and a list of works proposed to be undertaken is added, with notes on discussions between the public health and engineering authorities which led to the preparation of this list. Of the works actually carried out fuller details are given of three of the more important, which are typical of the schemes that can be attempted with a fair hope of success.

The works described in greater detail are (1) the filling of low-lying areas near the Jumna river; (2) the raising of the beds and the canalization of storm-water drains; and (3) the closure of the tail-distributary of the Western Jumna canal.

The Paper is accompanied by five sheets of drawings and nine photographs.

Paper No. 5200.

"The Fatigue of Concrete."

By WILLIAM FISHER CASSIE, M.Sc., Ph.D., Assoc. M. Inst. C.E.

(*Ordered by the Council to be published in abstract form.*)[1]

DURING the last decade increasing attention has been paid to the effect of repeated loading or vibration on concrete structures. The Paper is an attempt to gather together the principal conclusions reached by experimenters who have concentrated on the subject.

Small concrete specimens are always lacking in homogeneity, and in order to eliminate this defect it is essential that the specimens should be as large as is convenient.

Alternating stresses on concrete have generally been applied at a very much slower rate than has been used in fatigue-tests of other materials, and the duration of the experiment forces investigators to take into account the effect of ageing of the concrete specimen. It is necessary to allow the latter to attain the age of at least 6 months before testing.

In both young and old concretes during repeated loading the permanent deformations increase relative to the elastic deformations, but in young concrete this increase is much greater than in the older material. Further, of the deformation of the older concrete a very large proportion is elastic.

Tests have been carried out on cylinders and on beams of both plain and reinforced concrete, on cantilevers, on T-beams, and on doubly-reinforced beams. Various types of loading have been used.

It has been discovered by all experimenters that concrete subjected to repeated loading fails at a much lower unit stress than the ultimate static strength of the material. Van Ornum found that the specimen did not fail when the upper limit of the cycle of compressive stress was below 50 per cent. of the static ultimate strength. Clemmer arrived at a similar conclusion, and also determined that a considerably larger number of repetitions is required to cause failure in richer than in poorer mixes. Repeated loadings to some stress below the endurance-limit (from 50 to 55 per cent. of the static ultimate stress), results in an increased resistance to higher stresses as compared with the resistance of a previously unloaded specimen.

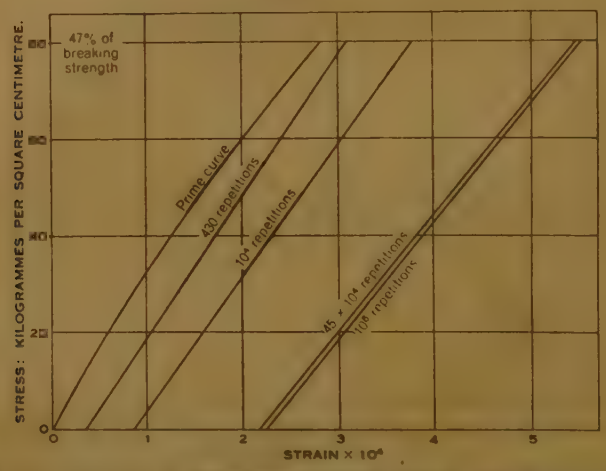
After a few repetitions of stress the relationship between stress and strain becomes linear, the projection on the strain axis of this straight line

[1] Copies of the Paper may be obtained on loan from the Loan Library of The Institution; a limited number of copies is also available, for retention by members, on application to the Secretary.

indicating the elastic deformation of the test-piece throughout the range of loading. The permanent deformation is indicated by the successive positions of this line relative to the zero of strain, or its "movement" in the direction of the strain-axis (*Fig. 1*).

When the upper limit of the cycle of repeated stress is less than the endurance-limit of the material, the stress–strain curve attains a straight-line form, and the curve ceases to shift in the direction of the strain-axis with increasing number of repetitions of stress; that is, the elastic and permanent deformations both attain a constant value, and the concrete behaves as an elastic material (*Fig. 1*). When the upper limit of repeated

Fig. 1.



STRESS–STRAIN CURVES FOR REPEATED LOADING BELOW THE ENDURANCE-LIMIT.
(Specimen P231, *Mehmel.*)

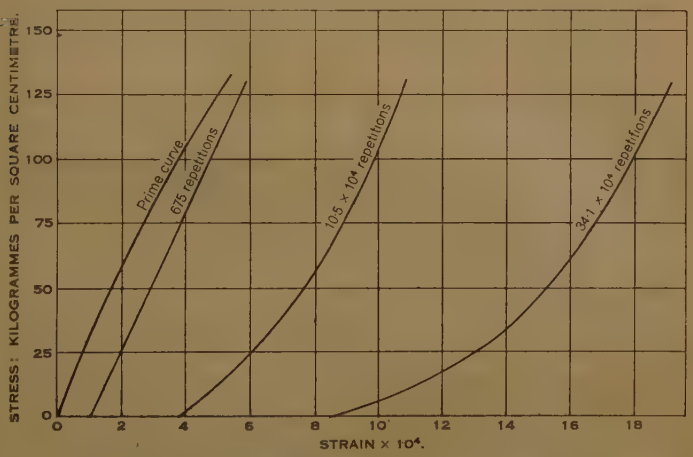
stress is greater than the endurance-limit of the material, both elastic and permanent deformations increase as the number of cycles of stress increases, and neither attains a constant value (*Fig. 2*).

The criterion of failure by fatigue is the shape of the stress–strain curve. As the ratio of the slope of the upper portion of the curve to that of the lower portion decreases below unity (the critical value), the concrete becomes increasingly fatigued (*Fig. 2*). Poisson's ratio decreases with increasing number of cycles of stress, and towards rupture attains the value of $\frac{1}{13}$ when the upper limit of the cycle is low, and about $\frac{1}{10}$ when the upper limit is high. Periods of rest from repeated loading have a beneficial, but probably merely temporary, effect.

The experiments analyzed may be divided into two groups according to the rate of repetition of the cycles of loading: (1) slow (up to 20 per minute); (2) fast (20 to 100 per minute). Very little data is available

on the relative effects of slow and fast loading, but it appears from the results examined that the slower rate of repetition results in larger deformations than accompany the same number of cycles applied rapidly. The effects of traffic-vibrations on concrete structures will probably be more accurately gauged from the results of high-frequency experiments (with repetitions of the order of several thousands per minute), than from those considered in the Paper. Such experiments have been carried out, but the results are not yet conclusive.

Fig. 2.



STRESS–STRAIN CURVES FOR REPEATED LOADING ABOVE THE ENDURANCE-LIMIT. (Specimen P251, *Mehmel.*)

Cracks appear under low stresses, and any internal stress which assists cracking (such as shrinkage-stress) should be kept as low as possible by proper curing. Cracks formed under repeated loading in reinforced beams do not increase in width indefinitely, but reach a constant value, provided that the stress in the steel does not exceed the elastic limit.

It is unlikely that a concrete beam or slab can be destroyed by indefinitely continued repeated loading, provided that the stresses in the steel and the concrete are not allowed to exceed the allowable stresses employed in the design. Undue cracking is unlikely to occur even under repeated loading if the stress in the steel does not approach the yield-point.

The Paper is accompanied by nine diagrams, from two of which the Figures in the Abstract have been prepared.

Paper No. 5199.

"The Economic Proportions of Steel in Concrete Sections Doubly Reinforced."

By TERENCE PATRICK O'SULLIVAN, B.Sc. (Eng.).

(*Ordered by the Council to be published in abstract form.*)[1]

THE design of a reinforced-concrete section of given dimensions subjected to a bending moment with or without an applied load is brought under review. This is limited to cases where tension occurs over part of the section, and where both tensile and compressive reinforcement must necessarily be provided.

The two methods in common use for the design of sections subjected to bending only are considered: namely (*a*) the normal method wherein maximum stresses in both steel and concrete are realized; and (*b*) the "equal steel top and bottom" method, wherein the concrete stresses are neglected.

In the same way consideration is given to the methods most used in practice for the solution of design for conditions involving a combination of bending moment with direct load. These are, (*a*) that most commonly in use, which assumes equal steel on each side of the section but at the same time limits the stresses within the maximum permissible values; and (*b*) that in which maximum stresses in both steel and concrete are realized.

The economics of each of these methods in their respective sphere is then brought into question, and the Author considers that an investigation should be made to discover their position in this respect.

A general analysis is therefore made of a given section subjected to a given eccentric load, and general equations are obtained for the areas of tensile and compressive steel required to reinforce the section within the allowable limits of stress. The minimum combined areas of reinforcement that may be employed are then obtained by evaluating the differential coefficients of the expressions for these areas, summing them and equating to zero. The variable chosen is the ratio of the depth of the neutral axis from the compressed edge to the effective depth, designated n_1.

From the resultant expression curves are plotted for values of n_1

[1] Copies of the Paper may be obtained on loan from the Loan Library of The Institution; a limited number of copies is also available, for retention by members, on application to the Secretary.

whereby this factor may be immediately determined from the known values of load, eccentricity, etc. Hence, upon substitution of this value in the equations for steel areas already determined, proportions may be obtained which give the least possible total amounts required.

It may be seen at once that the results thus obtained bear no direct relation with the established methods. In certain circumstances, however, one or other of these methods may be the economic solution.

Two typical examples are then given comparing the total amount of reinforcement required for designs carried out under the different suppositions.

By a simple adaptation the results obtained for a section subjected to a combination of bending and direct load are transformed, giving results for a section subjected to bending only. Further curves for n_1 are then prepared, which enable a solution to be obtained in a similar way.

Proceeding in the same way as for a section subjected to a combination of a bending moment and a direct load, an analysis is made for that subjected to bending together with direct tension. These equations also are adapted to the case of "bending only," so that it may be seen whether or not the results are in accordance with those obtained previously. The general conclusions are incorporated in the following summary:

(1) Where a section is subjected to a bending moment only, the reinforcement should in most cases be disposed in such a way that the maximum allowable tensile stress in the steel is realized simultaneously with the maximum compressive stress in the concrete.

(2) The main exception to the above is in the case of a section composed of ordinary 1 : 2 : 4 concrete and reinforced with a high-tensile steel, in which case the maximum allowable stress in the reinforcement will never be realized. The use of a high-tensile steel in such cases may therefore cease to be advantageous.

(3) Where a section is subjected to a combination of a direct compression together with a bending moment, neither the provision of equal reinforcement to each face nor the realization of maximum allowable stresses necessarily proves the economic solution. In such a case values of the compressive and the tensile areas required may be obtained from the information given in the Paper.

(4) Where a section is subjected to the combination of a tensile force together with a bending moment, the realization of maximum allowable stresses is the economic solution.

The Paper is accompanied by seven diagrams.

Paper No. 5178.

"Experiments on Stress-Distribution in Reinforced-Concrete Arches."

By Rhydwyn Harding Evans, M.Sc., Ph.D., Assoc. M. Inst. C.E., and Ivan Geoffrey Moore, Ph.D., Stud. Inst. C.E.

(Ordered by the Council to be published in abstract form.)[1]

Experiments have been carried out on three-hinged, two-hinged and hingeless arches, under symmetrical and unsymmetrical concentrated loads, with the object of investigating the distribution of stress, the magnitude of the horizontal thrust, and the deflexion at the crown. It is well known that the modulus of elasticity of concrete varies appreciably with the stress-intensity and with the time and method of loading, and that in applying the elastic theory to the analysis of an arch-rib it is assumed that the modulus of elasticity of concrete is constant for all sections of the arch and at all intensities of stress. The Authors considered it desirable to experiment on three-hinged arches before investigating the deformations in two-hinged and hingeless arches.

The span of the arch was 9 feet, the rise/span ratio 1/4, the thickness of the arch-rib 4 inches throughout, and the depth of the rib 9 inches at the springings and 6 inches at the crown. The hinges on the two-hinged and three-hinged arches were made in the form of steel knife-edges, whilst the springings were formed of cast-iron blocks supported on rollers and joined by a tie-rod, the strain in the latter being measured to determine the horizontal thrust. In the hingeless arches the reinforcing rods at the top and bottom of the arch were carried through the steel end blocks, and were secured to the latter by means of nuts. The end blocks were in turn bolted down on to heavy fabricated steel springings, and the springings were secured to rolled steel joists resting on rollers on the testing-machine table, two sets of rollers at right angles being provided to allow free movement in any direction. Measurements of the total strain and of the elastic strains were made at a number of sections. The stress-distribution was determined by measuring the strain with a 1-inch gauge-length roller extensometer in four directions; namely, parallel with the axis, at right angles to the axis, and at 45 degrees to the axis both ways. From these strain-

[1] The Paper and illustrations may be seen in the Institution Library.—Sec. Inst. C.E.

measurements the stresses were calculated, using the following relations :—

$$\text{longitudinal stress} = \frac{\left(e_1 + \frac{e_2}{m}\right)E}{1 - \frac{1}{m^2}}, \quad \text{cross or radial stress} = \frac{\left(e_2 + \frac{e_1}{m}\right)E}{1 - \frac{1}{m^2}},$$

$$\text{and shear stress} = \frac{\frac{1}{2}(e_{d1} - e_{d2})E}{1 + \frac{1}{m}},$$ where e_1 denotes the longitudinal strain, e_2 the cross or radial strain, e_{d1} and e_{d2} the diagonal strains, $\frac{1}{m}$ Poisson's ratio, and E the appropriate modulus of elasticity of the concrete.

The results obtained in the case of all the three-hinged arches indicate a satisfactory agreement between the calculated and observed values of the stresses, horizontal thrust and crown-deflexion. The action of a three-hinged arch-rib supported by rigid abutments conforms very closely to that estimated by the generally accepted theory of elastic structures. Under working stresses the maximum compressive stress observed in the concrete at any section of the arch agrees better with that estimated when it is assumed that the concrete in tension is assisting the tension-reinforcement. At the same time the observed tensile stress in the steel is nearer to that calculated including the tension in the concrete than that excluding that tension. The fact that the elastic theory including tension in the concrete gives better agreement on arches than on beams is probably due to the adoption of lower tensile stresses in the case of arches.

In the case of the two-hinged and hingeless arches the agreement between the observed and theoretical stresses and the horizontal thrust was satisfactory. In all cases it was found that the longitudinal stresses are easily the most important in the arch-ribs, and that the intensity of the shear stresses rarely exceeds the calculated value.

The deflexion at the crown in the case of the three-hinged and two-hinged arches was found in some cases to be appreciably higher than that calculated including tension in the concrete. It was, however, noticed that when a deflexion higher than that calculated was recorded it was accompanied by high bending stresses with tension on one side of the arch-rib. Calculations made which excluded tension in the concrete at points where tension was found produced a still higher theoretical value, showing that the observed value was between those calculated including and excluding the tension in the concrete.

The results are set out in a series of Tables and curves, and are compared with theory.

The Paper is accompanied by eleven diagrams and one photograph.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Committee on Earth Pressures.

A CONSIDERABLE extension of the programme of experimental work to be undertaken by this Committee has now been made possible by the financial support afforded by the four main-line railway companies and the London Passenger Transport Board, who have arranged to contribute jointly the sum of £1,000 per annum for 3 years towards the research. To supervise the extended programme of experimental work, an Experiments Panel of the Earth Pressures Committee has now been formed with the following personnel:—

RAYMOND CARPMAEL, O.B.E. (representing the Great Western Railway) (*Chairman*).
W. J. E. BINNIE, M.A.
G. M. BURT.
HERBERT CHATLEY, D.Sc.
Professor GILBERT COOK, D.Sc.
W. L. COWLEY.
J. R. DAVIDSON, C.M.G., M.Sc.
GEORGE ELLSON, O.B.E. (representing the Southern Railway).
W. H. GLANVILLE, D.Sc., Ph.D.
R. J. M. INGLIS (representing the London and North Eastern Railway).
V. A. M. ROBERTSON, M.C. (representing the London Passenger Transport Board).
R. E. STRADLING, C.B., M.C., Ph.D., D.Sc.
W. K. WALLACE (representing the London Midland and Scottish Railway).
F. E. WENTWORTH-SHEILDS, O.B.E.

In order that those engaged on the experimental work may be kept in touch with the many problems involving soil-mechanics that are met with by the railway companies, it has been arranged that one engineer from the staff of each Company will receive special training in soil-mechanics at the Building Research Station; he will thenceforth, while continuing his ordinary duties with the railway, give special attention to any soil-mechanics problems met with by the railway, and will keep in touch with the Building Research Station and maintain an exchange of information therewith. A similar arrangement is also being made with the Metropolitan Water Board.

The Experiments Panel has already held one meeting for the discussion of the present programme of experiments, and has visited the Building Research Station to inspect the work in progress.

THE UNDERGROUND WATER-SUPPLY OF LONDON.

A comprehensive study of this subject is given in a Report entitled "The Water Supply of the County of London from Underground Sources," which has been drawn up by Dr. Stevenson Buchan, and published as a Memoir of the Geological Survey of Great Britain.* The Memoir is divided into two sections, the first discussing the geology and hydrology of the strata underlying the County of London, and the second giving records of over one thousand wells.

The section on geology discusses in turn the superficial deposits and the solid formations of the London basin, which is formed essentially by the folding of the Cretaceous and Eocene beds into a major syncline. The principal underground reservoir is formed by the Chalk, which is underlain by the Upper Greensand and the relatively impermeable Gault.

The section on the hydrology of the London basin begins with a survey of the history of London's water-supply. The first underground supplies to be utilized were derived from shallow wells in the gravels of the river-terraces, which are fed by rain and in places by infiltration from the river, the water accumulating on the surface of the underlying impermeable London clay. Owing to the development of London, the water obtainable from the gravels became polluted, and is now only utilizable for some commercial purposes. After the gravel-water had been abandoned as the main source of underground water-supply, the Eocene beds (Reading Beds and Thanet Sand) became of importance as they contained a considerable amount of water gathered at their outcrops or derived from the Chalk. Owing to these beds standing higher at their outcrops than in the remainder of the London basin, and to the impermeable nature of the London Clay beneath them, wells piercing the clays and reaching the Eocene beds found water under pressure, which rose to the level of the potential water-table and in some instances overflowed as an artesian supply. In consequence of extensive utilization of the water of the Eocene sands they gradually became dry in most areas, and the wells had to be deepened into the Chalk, which now constitutes the most important water-bearing bed in London. The sands, however, still play a part in distributing water to certain areas of the basin, and there is considerable intercommunication of water between them and the Chalk.

The derivation of water-supplies from the Chalk is discussed very fully. It is explained that whilst the Chalk itself is permeable, a great deal of the water in the Chalk under London travels along cracks, planes of stratification, or other fissures. The Upper Chalk holds more free water than the Middle or Lower divisions; this fact has been demonstrated recently when wells deepened almost to the base of the Lower Chalk failed to increase their yield. It would appear that the amount of water available from the Chalk should be directly related to the amount of rain absorbed by the

* Published by H.M. Stationery office, price 6*s*. net.

outcrop, omitting for the moment any contribution from other strata; if this were so, and the rate of supply and demand balanced, the water in the reservoir would not be affected by pumping. If the demand exceeds the supply the level throughout the whole reservoir should be lowered, but this is not happening at the present day. The level in the centre of the reservoir is being lowered while surplus water continues to be emitted at certain points near the intake, as, for example, Ewell Springs. The factor which has most effect on the supply appears, therefore, to be the rate at which water travels from the catchment-area to the place where it is pumped out. The demand exceeds the supply which travels to the wells, but not necessarily the supply which is absorbed by the outcrop of the formation.

The areas from which the water in the Chalk is derived are discussed; it is explained that a great deal of water absorbed near the edge of the Chalk escarpment is thrown out as springs draining away from the London basin, and that some is lost by evaporation and surface-drainage and by utilization in wells for local supplies. On the north side of London there is a larger collecting area of Chalk than on the south; nevertheless a smaller amount of water passes to the centre of the basin, as the collecting area is farther from the centre and the gradient thereto is lower. Moreover, large quantities of water from the northern districts issue as the Chadwell and Amwell Springs, and therefore do not reach the central basin. The relation of the geological structure of the basin to the distribution of water is discussed.

Special consideration is given to the fall of water-levels and yields which has long been in progress, and maps are given showing the standing-water-levels in the London area in 1878, 1890–1900, 1911 and 1936. The fall in the water-level has rendered many wells useless as they are not now deep enough. In other instances wells have been deepened and pumps lowered, but there is a limit to the improvement that can be obtained by such means. Graphs of the fall in water-level show that the annual rate of fall is increasing; the falls in various districts are considered in detail by references to the records of wells given in the later part of the Memoir. The most important factors contributing to the lowering of the water-level at the centre of the basin are increased pumping, abstraction of water from the Chalk at its outcrop, more effective land-drainage and extensive building over the surface of the outcrop, and a recent period of low rainfall. The lowering of the water-level is beginning to cause pollution owing to percolation of salt water from the Thames estuary into the Chalk. It appears that the underground water-supply will continue to deteriorate for the reasons discussed above. Another potential danger to the supply is that there are now many abandoned wells which, owing to deterioration of the casings, may allow impure water from the gravel to travel down the wells into the Chalk. The records of certain deep boreholes show that there is little likelihood of a supplementary supply being obtained from the beds below the Chalk.

The catalogue of wells gives particulars of all known wells in the County of London, full particulars of strata passed through, etc., being given in the more interesting cases; the dimensions of the well, the Chalk- and water-levels, and the yield, are given in all cases, together with references to any other published information.

THE WORK OF THE BRITISH NON-FERROUS METALS RESEARCH ASSOCIATION.

The work of the British Non-Ferrous Metals Research Association covers the whole field of non-ferrous metallurgy, and is maintained by subscriptions from its members, comprising producers, manufacturers, or users of non-ferrous metals, together with a *pro-rata* grant from the Department of Scientific and Industrial Research. The Association is governed by a Council consisting of representatives of members and of Government Departments, and close contact is maintained between industrial needs and laboratory investigations by appointing special research sub-committees amongst interested members to govern each separate research.

The work at the Laboratories in Euston Street, N.W.1, is divided into three main branches; namely, Research, Development, and Information Bureau.

Research.

Amongst the many problems of which studies have already been completed are the production of sound strip-ingots, the soldering of metals, the wasting of copper locomotive firebox-stays, the corrosion of condenser-tubes, the properties of die-casting alloys, the electro-deposition of metals, and the frost-bursting of water-pipes. The following is a brief classified list of the researches at present in progress.

Melting and solidification.—The influence of gas atmospheres on the soundness and tensile properties of castings of aluminium alloys, nickel silver and bronze; the effect of grain-refinement and modification on the tensile properties and corrosion-resistance of cast aluminium alloys; segregation in ingots and castings.

Mechanical and Physical Properties.—Determination of the creep-characteristics and fatigue-resistance of lead and lead alloys at normal and high temperatures, with special reference to the effect of grain size and cold working; the tensile properties and creep-characteristics of copper alloys at high temperatures; the deep-drawing properties of metals (this in conjunction with the Institute of Automobile Engineers).

Corrosion and Protection.—The improvement of galvanized coatings; the corrosion of galvanized tanks and the use of galvanized tanks with copper pipes; the immersed and atmospheric corrosion of zinc coating; the corrosion-resistance of 70 : 30 cupro-nickel condenser-tubes, with

particular reference to the iron- and manganese-contents necessary in the metal; the effect of bacteria on corrosion of metals; the corrosion of copper service-pipes in different water-supplies, and the effect of minor constituents in the metal; the intercrystalline corrosion of lead; the soil-corrosion of copper, lead, and lead alloys (this has been under investigation for some years, and a comprehensive selection of specimens have been under field-test since 1931).

Miscellaneous.—The quantitative spectrographic analysis of metals; the electro-deposition of metals (further work), the production of bright nickel coatings, and methods for testing electro-deposited coatings; the weldability of non-ferrous metals; the surface-finish of rolled brass and copper sheet.

Reports on the work in progress are, in the first place, confidential to members, but researches of importance occasionally receive outside publication.

Development.

The Development Department deals with all questions relating to the industrial application of research results, and maintains close contact with members generally. The Department also deals with members' technical inquiries, which are not necessarily concerned with actual researches in progress, and which may or may not involve experimental work. Technical problems can frequently be solved by the application of results of researches previously obtained or by virtue of research, problems which would otherwise need long and many years' experience in investigating problems which may at first sight appear to be in a totally different field.

Information Bureau.

The Library, containing over 21,000 publications, is used by the Association's investigators, Development staff, and members alike. Not only does it provide references, abstracts, and translations for the use of the Association's permament staff, but it also sends, in response to requests from members, information varying from simple references to single Papers to complete bibliographies covering a whole subject. The Library also surveys very many periodicals published throughout the world, and issues each month a bulletin giving abstracts from all the important recent Papers dealing with non-ferrous metallurgy.

NOTE.—The Institution as a body is not responsible either for the statements made, or for the opinions expressed, in the Papers published.

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